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PRESTRESSED CONCRETE INSTITUTE

Journal of the prestressed concrete institute

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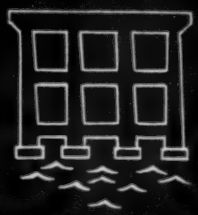

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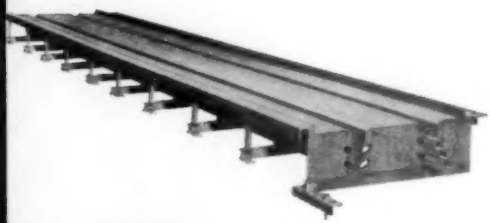

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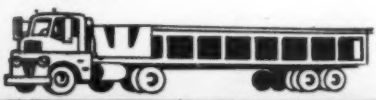

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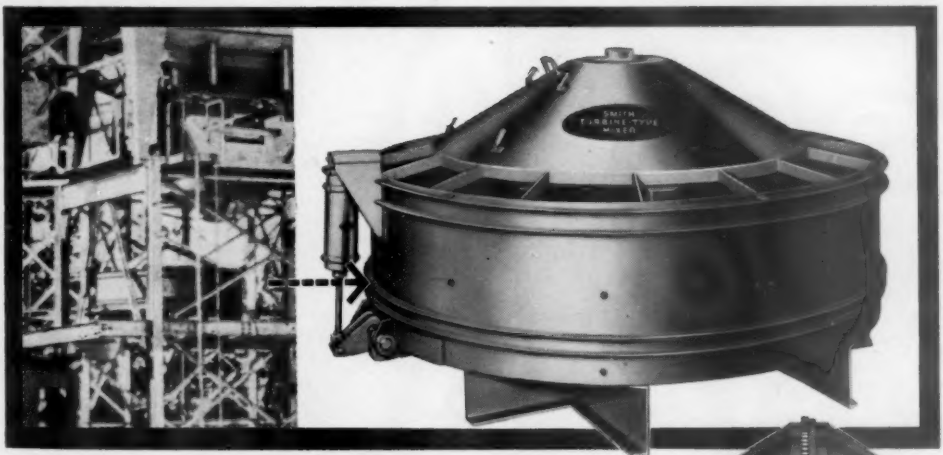

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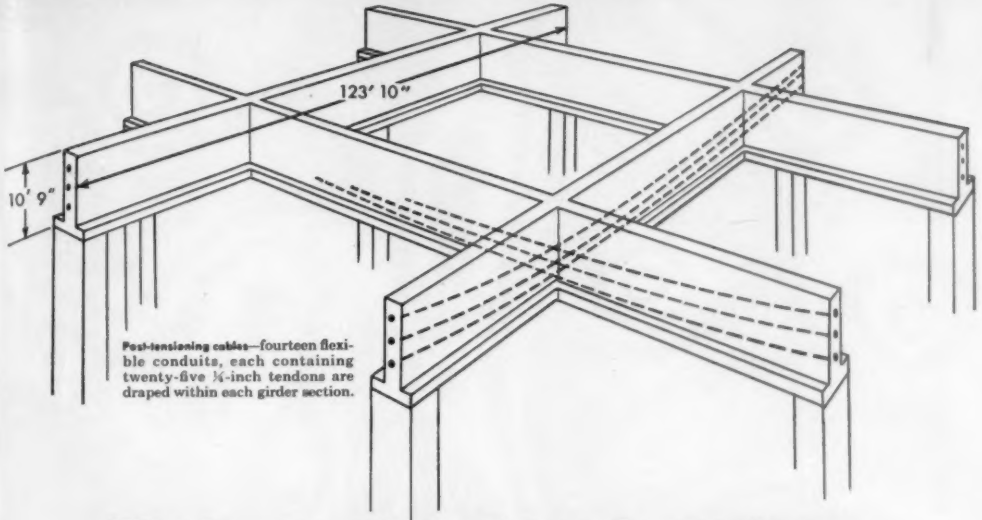

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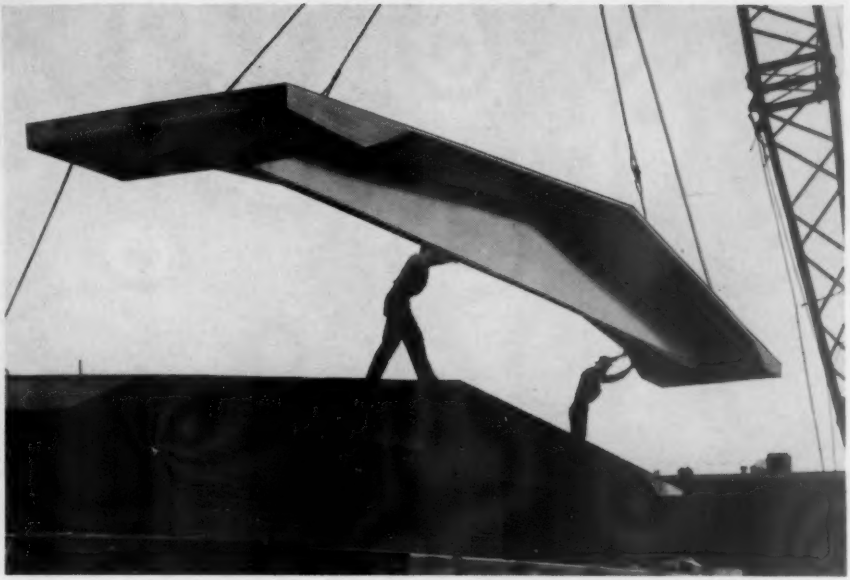

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New Pier 40, a massive concrete structure 800 feet square, is taking form in New York harbor. When completed in 1960, it will replace five older finger piers and will be one of the most efficient ship handling facilities in the country.

Substructure of the pier consists of a prestressed concrete slab deck resting on poured-in-place concrete beams and caps, supported on steel H-piles.

Six thousand prestressed slabs were used to form the deck. They measure 1 ft. thick, about 5 ft. wide and 20-25 ft. long. A large portion of this structural material was pretensioned with Laclede 7-wire prestress strand.

General contractor for all substructure work on the pier was Stock Construction Corporation, under contract with the Department of Marine and Aviation, City of New York, whose engineers are Roberts and Schaefer Company.

Pretensioned slabs for new Pier 40 were subcontracted to Corbetta-Precrete. They were cast in off-site prestressing beds and delivered to the harbor jobsite ready for immediate installation.

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(Above) New Flagship Rotterdam at Pier 40 on maiden voyage. Pier will have all-concrete superstructure. (Left) Typical slab being unloaded on pier deck. Most slabs measured 5′ x 20′ x 12″; weighed 7½ tons.

● For the new Holland-America Line Pier in New York, 6,021 prestressed concrete slabs were used in order to accommodate extremely heavy ultimate loads on a deck of minimum thickness (12″) resting on widely-spaced stringers (20′ apart). Design loads ran as high as 600 pounds psf.

Slab production and placement—a joint venture of CORBETTA CONSTRUCTION CO., INC. and PRECRETE, INC. (both of New York)—continued at a high rate through an extremely cold Winter. Dependable 'Incor' High Early Strength Cement was used in an 8-bag, steam-cured, 5000-psi concrete, with stress-release strength specified at 3500 psi. 285 special 6000-psi slabs were released at 4650 psi. Release strengths were generally attained in 15 hours. Joints were concreted with 3750-psi 'Incor'® concrete, permitting loaded trucks to ride over deck sections in two days.

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GUEST EDITORIAL

The prestressed concrete industry has become an important part of the building and construction industry in this country since its initiation less than a decade ago. Few industries can boast of equal accomplishment in so short a time. The success of this infant and yet major industry must be attributed to the fact that it met the needs of the structural designer in furnishing economical and safe structural components for bridges and commercial buildings, large and small.

The University of Florida has made many significant contributions to the prestressed concrete industry which had its infancy and early growth in Florida. (Florida, I believe, still leads all the other states in the number of prestressing plants.) Research in prestressed concrete at the University of Florida, through the cooperative efforts of our State Road Department and the Prestressed Concrete Institute, has been going on for almost ten years. The results of these investigations are being made public through our own bulletins, the JOURNAL of the Prestressed Concrete Institute, and other national publications, providing a better understanding among engineers and architects, of the behavior of prestressed concrete members. In cooperation with the Prestressed Concrete Institute we have held three national short courses which were attended by about one thousand professional people from various parts of the United States as well as other countries. Many of our graduates are presently employed, some in key positions, in the prestressed concrete industry all over the country and abroad.

The prestressed concrete industry is still young. It has a bright and challenging future. Prestressed Concrete Institute members and officers must meet this responsibility by continued efforts in solving the many problems in design, application, and quality control. The success of the past promises to be only the beginning for the industry.

Engineering schools have a responsibility for educating students so that they will be able to utilize this new technology in a practical manner upon graduation. The industry has a responsibility to support research in engineering schools so that new knowledge is generated and that educators keep continually abreast of new developments as they arise.

EDITOR'S COMMENTS

Dean Joseph Weil, our Guest Editorial Writer, is known nationally as one of the country's leading proponents of nuclear energy. On the one hand a giant in scientific and technical accomplishments as Director of the Engineering and Industrial Experiment Station which produced that ingenious radio proximity fuse, the hurricane network for the Weather Bureau and innumerable defense devices, Dean Weil is equally a titan of education, dispensing his powerhouse of brains and energy to all branches of engineering, the results of which add up to legends. And if no more than his vast contributions to the prestressing industry be of interest to you, that alone would merit his national acclaim as pioneer and top contributor. For the Dean perceived early the future and magic of prestressed concrete, opened the Experiment Station to the first research program projected by PCI, took up a large portion of its costs and has since sparked three national short courses for professionals. The last paragraph in his editorial should be displayed in every school of engineering and in every prestresser's office.

• • •

Before you start the first article, take a relaxed posture, your feet up on a desk, a bottle of milk, pop or Three Star by your side, an ash tray within aiming distance, then meet Richard Gensert, consulting engineer, talented, creative and imaginative as expressed by his "Shape of Prestressed Concrete."

• • •

Still relaxed? Now meet Milo Ketchum of Denver, rich in experience and accomplishments, and in creative design. Milo writes about a prestressed tapered roof joist, a new product which might, I believe, very easily make a grand slam in universal use for its efficacy, and aesthetic appearance.

• • •

Our next paper presents a recent application of prestressing in building construction which combines the advantages of a folded plate concrete roof with post-tensioning in place—the author, James Owen Power, member Dignum Associates, Coral Gables, Fla.

• • •

E. A. L. Smith, retired chief mechanical engineer, Raymond International, Inc., draws upon long and important experience to tell us about, "Tension in Concrete Piles During Driving."

• • •

Harry Edwards, pioneer in prestressing—formerly Chairman PCI Specifications Committee, Member PCI Building Code Committee; President Leap Associates, Lakeland, Fla., presents a most interesting and provocative paper, "Connections by Bond."

• • •

"Corrosion Tests on Prestressed Concrete Wire and Strand," by Howard J. Godfrey, John A. Roebling's Sons division, is based on the most responsible knowledge, research and tests available.

• • •

Murray Schupack, partner, Schupack & Zollman, nationally famous consulting engineers, presents one of his typical top-notch bridge designs for a structure built in Illinois.

• • •

An original paper, "Composite Construction for Continuity," by Eivind Hognestad, Alan H. Mattock and P. H. Kaar, Portland Cement Association, reflects the optimum in equipment owned by their Research and Development Division and the outstanding scientific caliber of their staff.

• • •

Ross H. Bryan, consulting engineer, Nashville, Tenn., gives us a most valuable paper, "The Effect of Bundled Strand on the Performance of Prestressed Beams," based on his own tests which I recommend to the world.

The Shape of Prestressed Concrete

By R. M. Gensert*

Illustrations by Robert C. Gaede, Architect

The author sees an individual personality developing in prestressed concrete as the industry gains maturity and attacks new problems. Mr. Gensert shows how many prestressing problems have been solved by the trial and error method.

Unfortunately, the advent of a new material confronts us with the problem of finding its unique personality. This problem comes about through the effort to substitute for an established material. In the case of concrete, its first uses were imitations of heavy masonry arches. But as designers and builders became more familiar with the properties of concrete, we saw it used linearly in beam and column construction and eventually in the flat slab and thin shell where it becomes a three-dimensional medium.

Today, the precast, prestressed industry is faced with the same developmental problem—that of replacing steel and wood beams and columns, with precast prestressed members. We all know that this is another step in the maturing of a material, yet we need to be aware of the fact that this material undoubtedly has its own form and function. The discovery and exploitation of the true character of prestressed concrete would not only change the status of this construction from merely an alternate design for other materials, but would project it far beyond into a form of construction wherein it would have its individual personality and advantages and no competition.

This problem is also evident in the young sister material, structural aluminum. One particular instance was a competition between structural steel and structural aluminum wherein a tower was to be built for the least cost—weight and maintenance not to be considered. The aluminum interests employed a staff of specialists to attack the problem, wherein aluminum angles and wide flange shapes were sized for the maximum stresses that the aluminum would be subjected to. In spite of the fact that nothing was overdesigned, the steel structure cost less. Still determined to break the cost barrier, the problem was reattacked by employing thin, extruded, fluted plates arranged to form a large cylinder. This solution not only beat the cost of the steel frame but produced a structure that was very pleasing to the eye. Just think what could be accomplished in the case of concrete! When precast, prestressed concrete shaped as I-sections and wide-flanged sections is already competitive with structural steel in many cases, imagine the cost savings possible by taking advantage of the one great property concrete has—that is the ability to be shaped into thin, curved sections where every

*Gensert, Williams & Associates
Consulting Structural Engineers; Cleveland, Ohio

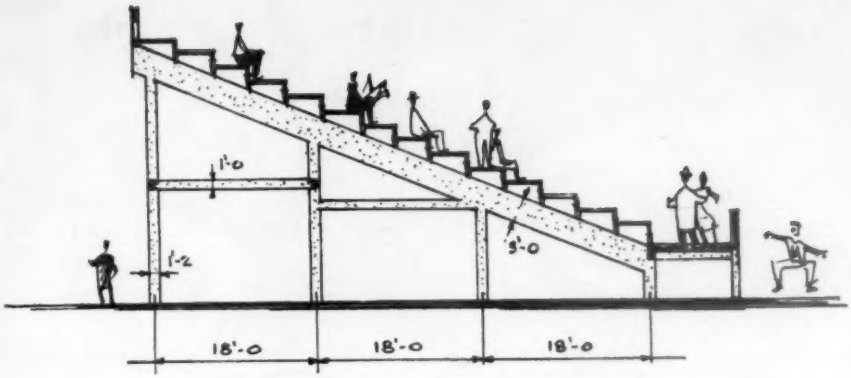


FIG 1

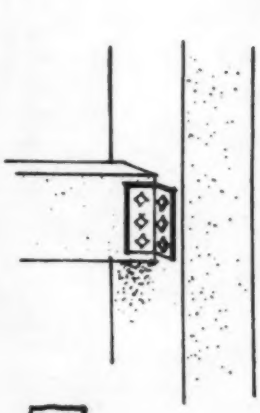


FIG 2

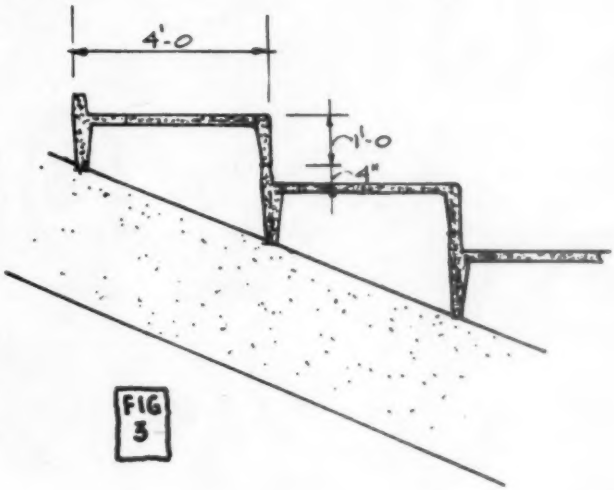


FIG 3

portion of the material is stressed to its limit.

In Structures

By using precast, prestressed concrete in much the same way as other materials, we are faced with problems. For instance, Fig. 1 shows a precast bleacher that was designed along the same lines as a structural steel bleacher. Struts were incorporated to brace the columns and were connected as shown in Fig. 2, leaving clip angles and bolts for everyone to view. The platforms of this bleacher were adapted channel sections, producing voids between supporting beams as well as a very cluttered ceiling. See Fig. 3. By using concrete naturally, a row of columns could have been replaced with a cantilevered extension of the beam, and by slightly increasing the column sizes, the struts could have been eliminated. See Figs. 4 and 5.

The deck, too, could have been cleaned up by the use of prestressed

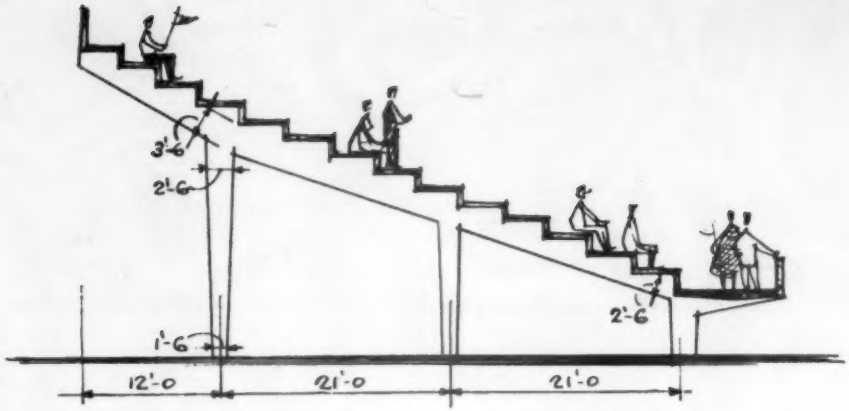


FIG. 4

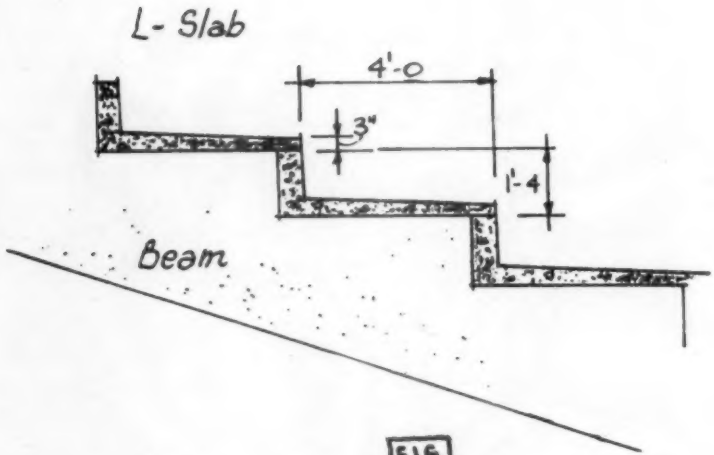


FIG. 5

L-shaped slabs as was done for a grandstand in Columbus, Ohio. The L-shaped slabs for the Columbus Trotting Association were constructed by the Concrete Masonry Corporation in Elyria, Ohio. Fig. 6 is a picture taken during the load test of one of these slabs when the live load was twice the design value.

In Connections

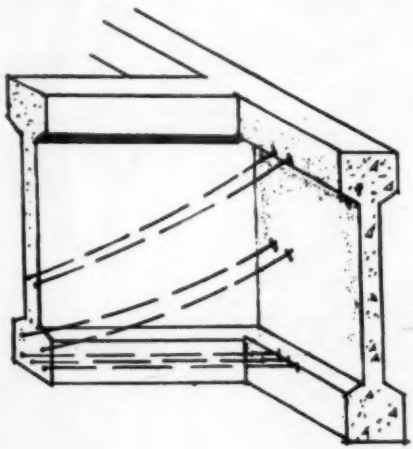
The problem of connecting precast elements is a substantial one, not only insofar as costs are concerned, but also from the standpoint of appearance. The beam connections shown in Fig. 7, made by means of partial prestressing, are far more desirable appearancewise

Fig. 6

than the beam connection shown in Fig. 8, where clip angles and bolts are used.

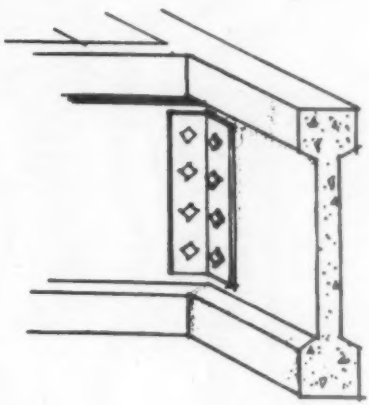
In Bending Members

Again, let us evaluate the prestressed concrete truss shown in Fig. 9 where the shape was determined by its steel prototype. Compare this in both cost and appearance with that of Fig. 10 where precast shell segments are tensioned together to make a shell structure of larger span. It is obvious that the shell also has a greater resistance to buckling whereas the truss will require bracing members to attain stability. Assuming that the curved shell roof would not architecturally replace the flat roof of the truss, it



PRECAST CONCRETE ELEMENTS CONNECTED AS IF THEY WERE MADE OF STRUCTURAL STEEL

FIG. 8

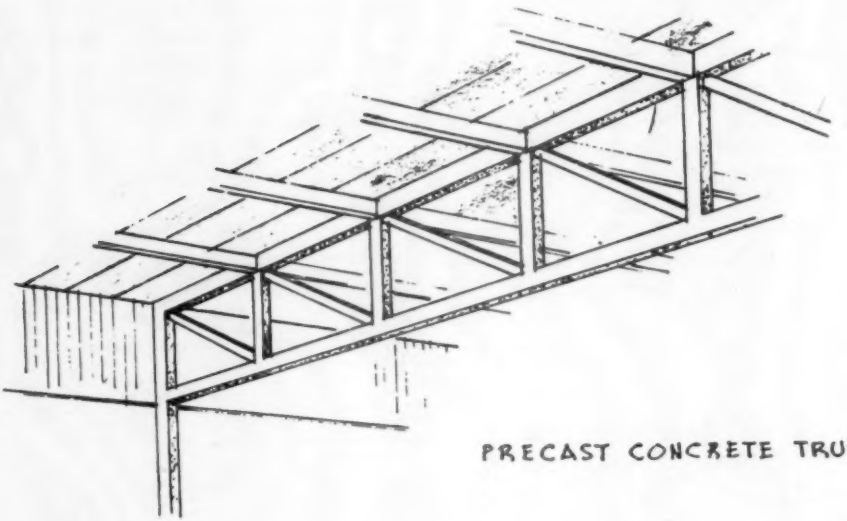


PRECAST CONCRETE ELEMENTS CONNECTED BY POST-TENSIONING

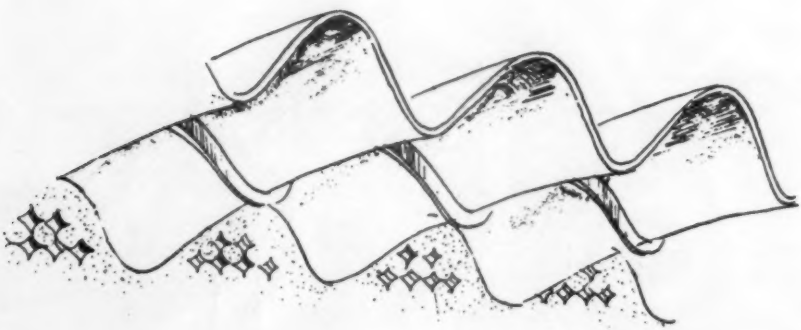
FIG. 7

may be more expeditious to use a precast, prestressed truss without the cumbersome diagonal members. This later system, the Vierendeel truss, is more appropriate for the flexural capabilities of prestressed concrete, Fig. 11.

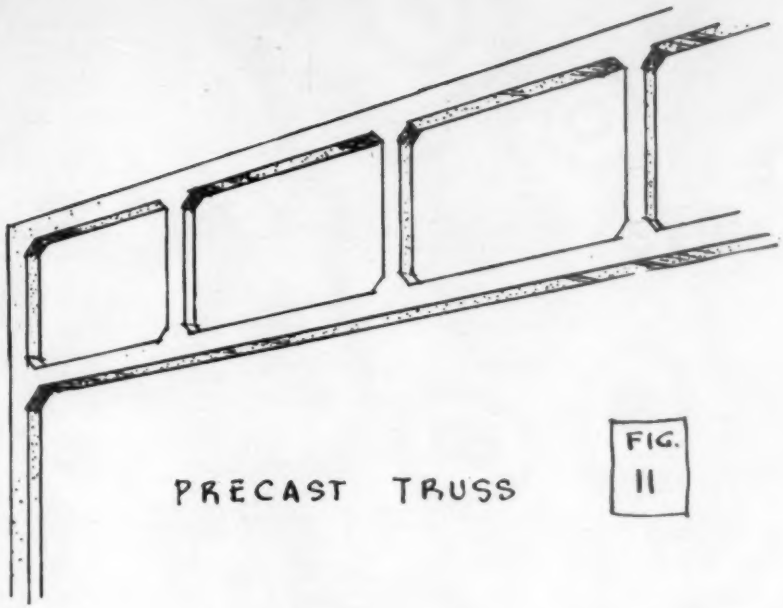
Fig. 12 shows the familiar prestressed concrete simple beam, the cables of which are draped so as to eliminate the eccentric moment at the end of the beam where the moment from the applied loads is zero. It is not only convenient construction to keep the tendons straight but we also have the advantage of eliminating frictional losses. We could make the beam deeper at the ends than in the center, as in Fig. 13, to eliminate the eccentric prestress moment. The resulting beam takes on an arch-like appearance and thus gives us a clue to one facet of the prestress personality. Fig. 14 shows such a prestressed beam in use.



PRECAST CONCRETE TRUSS FIG. 9



PRECAST CONCRETE SHELL FIG. 10



PRECAST TRUSS

FIG. 11

In Continuity

Cast-in-place construction presents a problem in prestressing where continuity is involved since frictional losses may be a prime factor in construction. Fig. 15 shows a typical solution where the tendons are kept straight and the concrete section varies throughout the span. Thus prestressed concrete has a definite profile whether it is precast or cast-in-place.

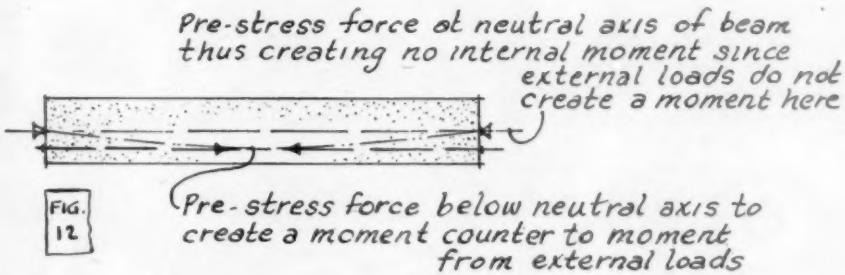
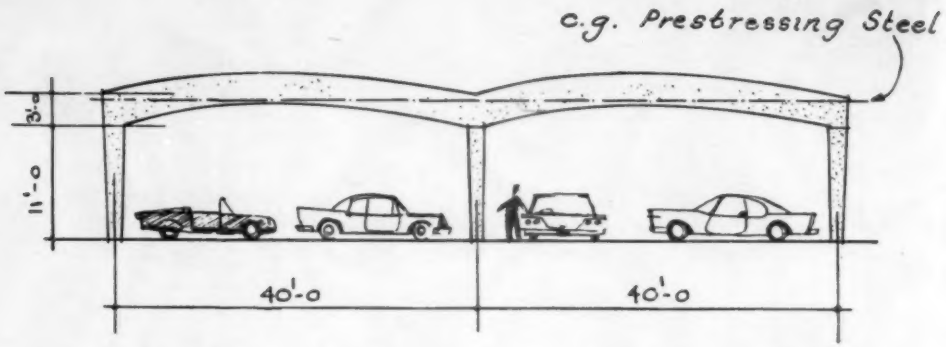
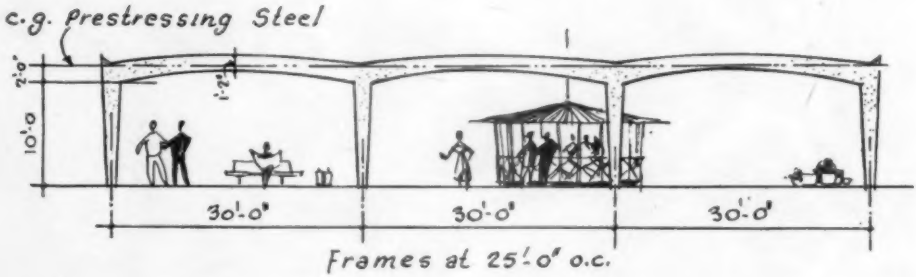

Pre-stress force at neutral axis of beam thus creating no internal moment since external loads do not create a moment here
Pre-stress force below neutral axis to create a moment counter to moment from external loads

FIG. 12

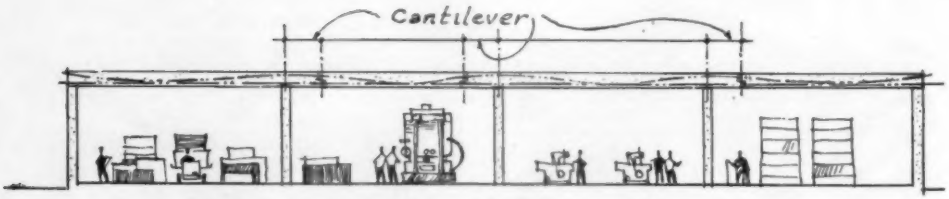
FIG. 13



PRECAST - PRETENSIONED FRAMES FIG. 14

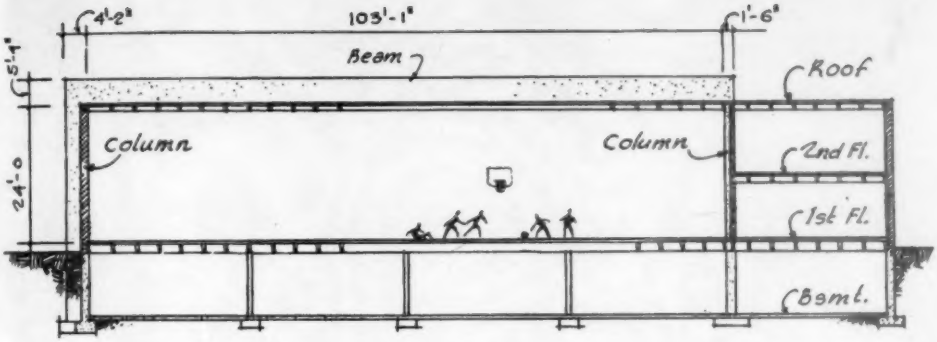


POURED - IN - PLACE - POST - TENSIONED FRAMES FIG. 15



ARTICULATED STRUCTURE WITH DRAPED CABLES FIG. 16

We spend much time and effort worrying about the loss in prestress and joint efficiency in continuous structures. In actuality, the continuous beam is not as efficient as an articulated beam system, where negative moments from the cantilevers reduce the adjacent positive moments. Fortunately, the draped strands for anchoring positive reinforcing are in ideal position for reinforcing the relatively short can-



UPSET - PRESTRESSED CONCRETE BEAMS FIG. 17

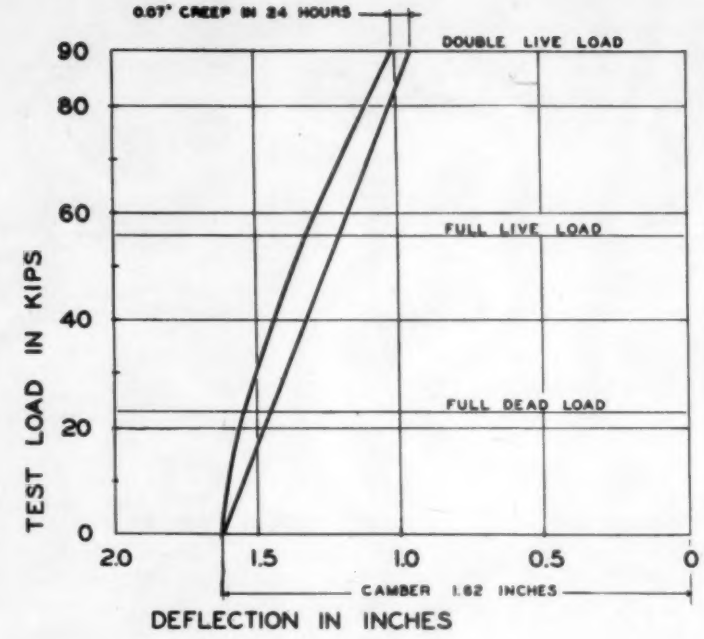
tilevers. Fig. 16 illustrates the details of such a structural mechanism.

In large auditoriums and theaters, the depth of roof beams becomes excessive and represents an appreciable expense when one considers the amount of extra wall construction required to enclose the space above the ceiling line. Prestressed concrete, unlike other materials, exhibits an inherent quality of resistance to buckling when the tendons are bonded. This was taken advantage of in a school auditorium for St. Stanislaus in Cleveland, Ohio. (Nicholas Lesko, Architect) Fig. 17 shows the beams upset above the roof deck. Accordingly, these compression flanges were laterally unsupported. A load test on a similar project for the St. Vitus School Auditorium in Cleveland (John F. Lipaj, Architect) substantiated this theory. The deflection curves for the St. Vitus beam are shown in Fig. 18.

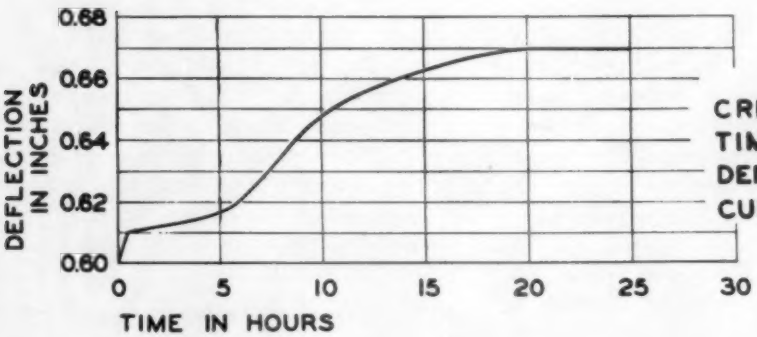
Generally, the compression flanges of prestressed concrete beams require a larger area than the tension flanges. Thus, the usual system of beams for bridge construction may be wasteful both in the amount of concrete which is used as well as the dead load that is produced. See Fig. 19. An alternate solution to this might be one in which the top flange is wide enough to resist compressive buckling stresses, whereas the tension flange is a minimum. The addition of web material may also be desirable for shear and torsional resistance. See Fig. 20. There is no doubt that form costs for the bridge beams shown in Fig. 20 will be high. However, this is a problem of manufacture that is no greater than the many other once-insurmountable obstacles that have been solved by the industry.

In Compression Members

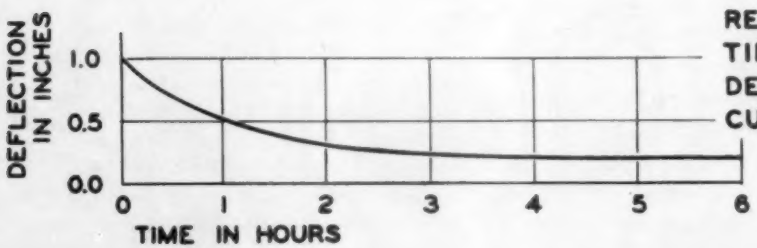
The first lift in a Youtz-Slick lift-slab system of construction requires a cantilevered column. Subsequent lifts are limited to a lesser degree, but the column strength is still a function of its tendency to buckle. In multi-story buildings, the cost of steel columns sets quite a handicap on the lift-slab method. While conventional concrete columns are no match for the elastic steel columns, the solution to this problem can lie in prestressing. Here again we build



LOAD DEFLECTION CURVE



CREEP TIME DEFLECTION CURVE

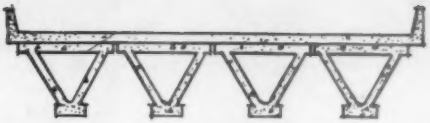


RECOVERY TIME DEFLECTION CURVE

FIG. 18

TYPICAL PRECAST BEAMS FOR A BRIDGE

FIG 19



PROPOSED PRECAST BEAMS FOR A BRIDGE

FIG 20

a stabilizing force into the member, and virtually eliminate the buckling of unstable bending and compression members.

In Shell Roofs

The economies that have been produced with the cast-in-place, folded plate, and the barrel shell should be available to the precast business as well. Figs. 21 and 22 show two possibilities in this regard. Perhaps the biggest problem is obtaining some degree of transverse continuity. Fig. 21 shows one method of obtaining this transverse connection by the use of high-tensile bolts. Fig. 22 shows another method wherein transverse ribs are post-tensioned together for the purpose of making this connection. A typical arrangement for the longitudinal steel in prestressed thin shells is illustrated in Fig. 23.

In Conclusion

The coming of age of any construction method is a slow and tedious process that often diverges in many directions before it realizes its final position in the design and construction field. This is good. Mistakes are necessary steps in the learning process that serve as measuring devices for the final coup. Such an accomplishment, however, cannot be done by one individual or even one segment of the industry. Prospective building owners, manufacturers of prestressed concrete products, contractors, architects, engineers, and even municipal building departments must all work together in an effort to understand

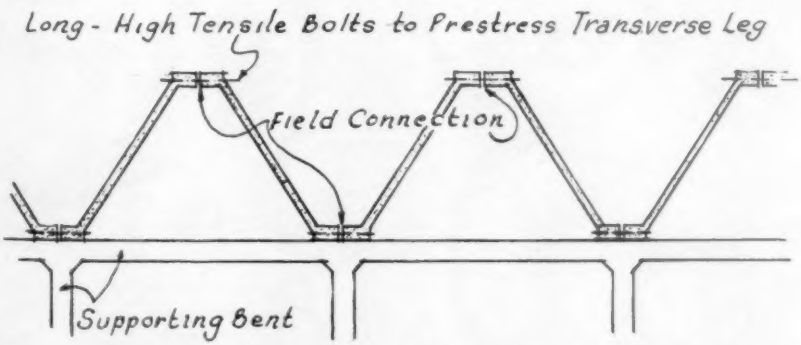


FIG. 21

PRECAST - PRESTRESSED ELEMENTS OF A FOLDED PLATE

problems, limitations, and new uses for this construction. In the Cleveland area we have been most fortunate to have at least two competent concrete products plants that have been willing to interrupt their production lines in order to manufacture some non-typical products. This pioneering spirit has resulted in the construction of arches, shells, domes, vaults, and many other special building and bridge members.

Before closing this dissertation on the natural form and function of another man's."

prestressed concrete, I must admit that these examples are not intended to be the optimum solutions of the precast, prestressed medium. They are merely simple, forthright solutions to specific problems rather that problems derived from preconceived solutions—to wit we might consider the advice of Ralph Waldo Emerson: "Imagination cannot go above its model. The imitator dooms himself to hopeless mediocrity . . . In the imitator something else is natural, and he bereaves himself of his own beauty, to come short of

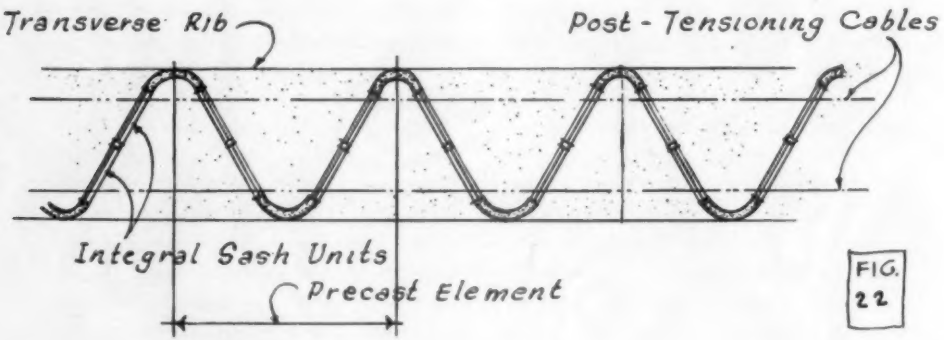
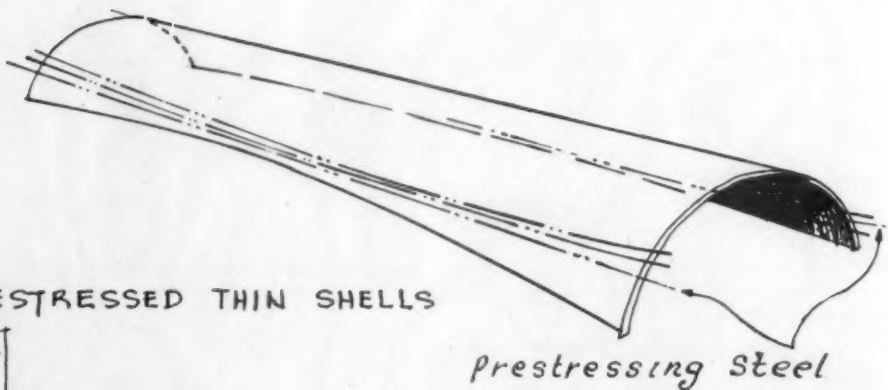


FIG. 22



PRESTRESSED THIN SHELLS

FIG. 23

Prestressed Tapered Joists for Roofs

Milo S. Ketchum

Ketchum, Konkel and Hastings
Consulting Engineers
Denver, Colorado

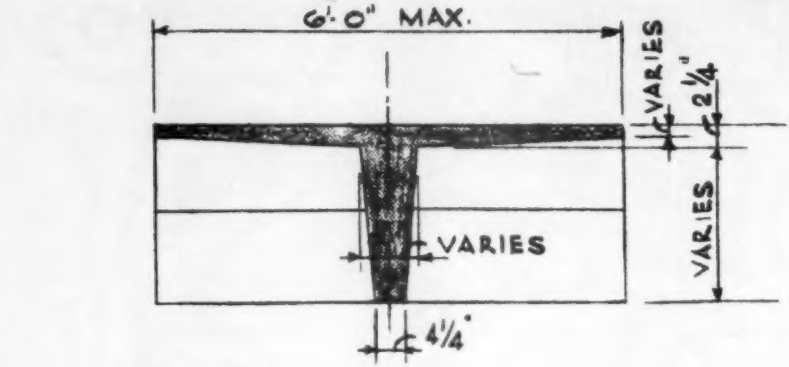
SYNOPSIS

Maximum economy of material in prestressed concrete roof construction indicates that the joists be sloped to a maximum depth at the peak with an absolute minimum depth at the supports. Using this criterion, a series of single tee joists were developed for use with modular forms. The area at the support is increased by increasing the width of the flange to the full width of the joist. Some of the problems of design are discussed.

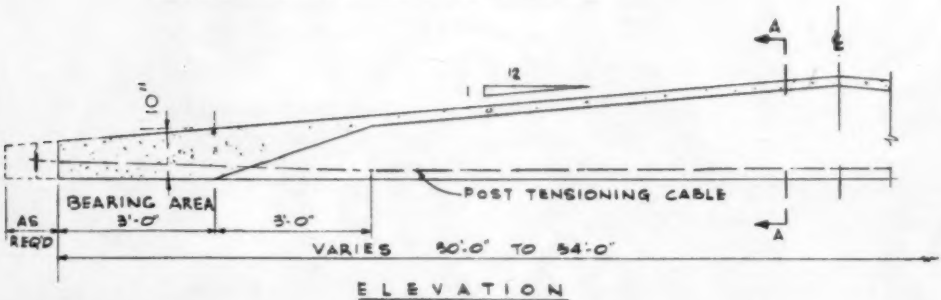
In the competitive market that now exists for roof construction for single story buildings, it is absolutely necessary to have a minimum of materials combined with maximum utilization of forms. At the same time the structure must be flexible in its adaptation to dimensions in the field and must have that intangible value of appearance which will create consumer demand.

These basic requirements have led to the design and development of a pre-cast joist called the Flex-Tee by the manufacturer, Flexforms, Incorporated of Englewood, Colorado. A sketch of a typical unit is shown in Fig. 1. The span of joists varies from 24 feet to 54 feet for a 45 psf roof load. Over 54 ft. and up to 60 ft. slightly less live load can be carried. The slope of the top of the joists is a constant 1 to 12 pitch. The end depth varies from 10 in. to 7 in. depending upon the point of anchorage. The slab can be cantilevered any reasonable distance beyond the anchorage. The minimum width of the web at the bottom of the joist is 4.25 in. and increases toward the flange. The slope of each side of the web is a constant 1 to 10 so that for the greatest depth the width of the web is about 10.25 in.

At a point about 6 ft. from the end of the joist the flange increases in depth so that it is the full depth of the member at a point 3 ft. from the end. The area of concrete available for shear and diagonal tension at the support is, therefore, increased substantially at the support. Additional advantages of these joists are that no filler blocks are required at the ends as for normal joist construction, and the location of the support point may be varied within a range



SECTION A-A



ELEVATION

PRESTRESSED TAPERED JOISTS FOR ROOFS

FIG. 1

of 3 ft. to fit unusual architectural details.

The prestressing of this unit is accomplished by a single post-tensioning cable as shown in the sketch. In addition, reinforcing steel is required at the support for longitudinal tension and for diagonal tension. The Prescon System has been used for these joists. Either pretensioning or post-tensioning may be used. The initial design has been made for post tensioning because a pretensioning bed is not required, and it has other advantages to be noted.

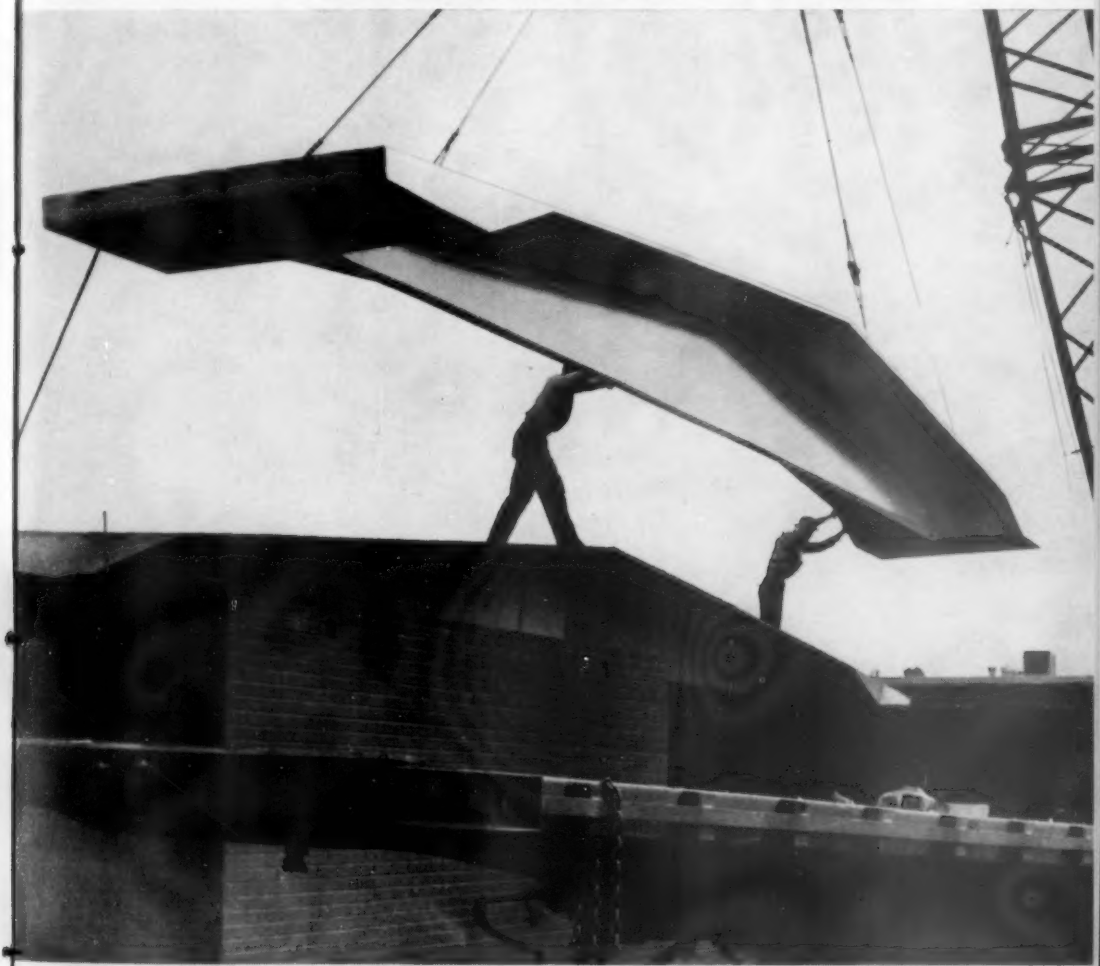
The highest stress and the most critical design problem in this structure is in the analysis of shear, diagonal tension and longitudinal reinforcement near the support where the depth is decreasing at a greater rate than the bending moment. The stresses at the center are quite low due to the generous depth. To take full advantage of the design, the clause in the ACI-ASCE Prestressed Concrete Recommendations allowing criteria to be set by actual load tests should be invoked. Such a series of tests is now contemplated. Although the design is now satisfactory with such tests, it is felt that additional economy can be achieved.

A key to the economy of any concrete structural system is the method of forming. With this type of joist, all spans have precisely the

same end detail. The extra depth is achieved through the slope of the top member. The width of the cantilever top slab may be changed as desired to accommodate different conditions.

Steel forms are used. The form assembly for each end is twelve feet long from the inner end of the bearing pad. In order to increase the span, two units each three feet long are inserted between the end units. Variation less than this amount can also be accommodated by locating the bearing points for support of the joists. Thus, any span can be accommodated from 24 ft. to 54 ft. or greater for certain live load conditions. The steel form is constructed in a single unit so side forms are not removed prior to removal of the joist. This speeds fabrication. The wide end detail of these joists increases the torsional stiffness and makes erection somewhat easier than conventional joists.

Erection of 56 ft. Flex-Tees on building in Lakewood, Colo. Units produced by Flexforms, Inc. Nixon & Jones, architects. Johnson & Voiland, structural engineers.



Control of camber is not a problem in these joists due to the relatively low stresses in the middle of the span. In addition, the post-tensioning permits tensioning in stages so that the concrete is at its optimum strength and modulus of elasticity. Measurements taken on 15 identical units showed a maximum variation of 3/8 in. even though the initial post-tensioning was applied over a range from 2500 psi to 4500 psi cylinder strengths. The total camber averaged about 2 in. Sixty per cent of tensioning only is necessary for handling so that units can be removed from the forms for greater utilization.

Preliminary to the design of these joists, load tests were made on developmental models by the United States Testing Company, Inc. of Denver. Span of joists was 32 ft., 2 in. Minimum widths of the web at the bottom was 3 in. Initial cracks occurred at a load of 60 psf at a point about 7 ft. from one end. A total of 70 psf was applied to this joist. The maximum deflection was 1.778 in. and the final deflection 24 hours after release of load was 0.063 inches. Cracks were not visible after load was released.

Those joists were developed by Flexforms, Inc. of Englewood, Colorado, I. C. Peterson, President, who is making this product available nationally through local fabrication. Ketchum, Konkel and Hastings were engaged to review the final design.

Folded Plate Concrete Roof Is Post-Tensioned in Place

By James Owen Power,

Chief Engineer,
Dignum Associates,
Coral Gables, Fla.

A recent application of prestressing in building construction combines the advantages of a folded plate concrete roof with post-tensioning in place. The project is the Cloverleaf Lanes Bowling Alley in Dade County, Florida, at the southern entrance to the Sunshine Parkway. The design problem required provision of a 34,000 sq. ft. area—50 bowling lanes—unobstructed by columns. The structural system was required to be economical and attractive in appearance. Since construction time was limited, no system would be acceptable which could not be completed rapidly.

The solution consisted of a corrugated concrete slab, spanning approximately 120 ft. and extending transversely 286 ft. for the length of the building. The overall depth is 6 ft.; the horizontal module is 22 ft. Top and bottom flanges are 8 in. thick and the inclined webs are 5 in.

This folded slab is supported at

BELOW—Aerial view of Cloverleaf Lanes showing folded plate roof.

ABOVE—Construction view of roof section showing temporary rods and turnbuckles. Note precast concrete window grilles.

the west end of the span on concrete columns which support the center of the bottom flange. Precast concrete frames cast to the shape of the corrugation were used as window grilles as well as diaphragms. At the east end, the roof spans to a continuous 12 in. block wall framed with concrete columns and beams.

Each interior corrugation is reinforced with 14 Roebling cables; 8 of these are placed straight in the bottom flange and 3 are placed parabolically in each web. All cables are composed of 12 wires, .276 in. in diameter. These wires are anchored in Freyssinet cones and grouted after tensioning. Mild steel was used to

BELOW—Construction view from interior showing how concrete window grilles were used as diaphragms. Note forming still in place between adjacent pairs of corrugations. Prestressed double tees are in place in left and right background.

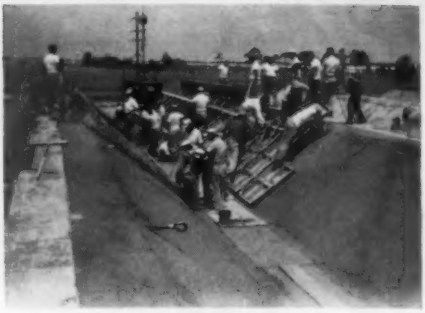
provide for temperature stresses and load distribution in the transverse direction. Special mild steel reinforcing was added in the top corrugation to provide for construction loads which will be discussed later.

At the north and south ends of the building, the bottom flange of the corrugation rests on a continuous block wall and extends outward to provide an overhang. The half-bays adjacent to these walls were not post-tensioned but reinforced with mild steel. The roof design followed the requirements of "The Tentative Recommendations for Prestressed Concrete of the ACI-ASCE Joint Committee 323". Valuable design aids were ASCE Proceedings Paper 1580 "Design of Folded Plates" by Howard Simpson and Y. Guyon's "Prestressed Concrete". The latter was used as a basis for the end anchorage design.

Economic considerations made a maximum reuse of forms essential. Since this would require the deshoring of a portion of the roof before pouring the adjacent section, it was necessary to devise temporary means of providing lateral support at longitudinal construction joints. This was accomplished in the following manner: Construction joints were limited in location to the upper flange and permitted no closer together than alternate bays; that is, it was required that at least two bays be poured at one time. A longitudinal open joint 8 in. wide was provided along the centerline of the top flange between adjacent sections. This joint was

ABOVE—Rear wall of bowling lanes showing portable jacking unit in use on roof.

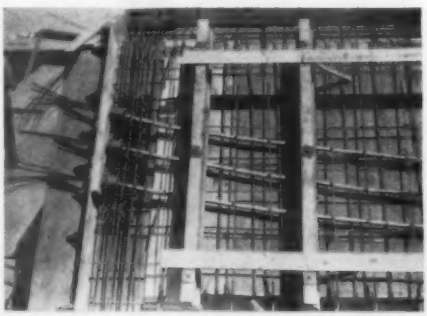
poured in after both sections were tensioned and shoring was removed. Temporary lateral support was provided by means of steel rods and turnbuckles spaced at 20 ft. on center and running from exterior top flange to inserts in the interior top flange. The exterior top half flange was reinforced to span horizontally between the rods. After adjacent sections had been deshored and the joint between them had been poured, these rods were removed.

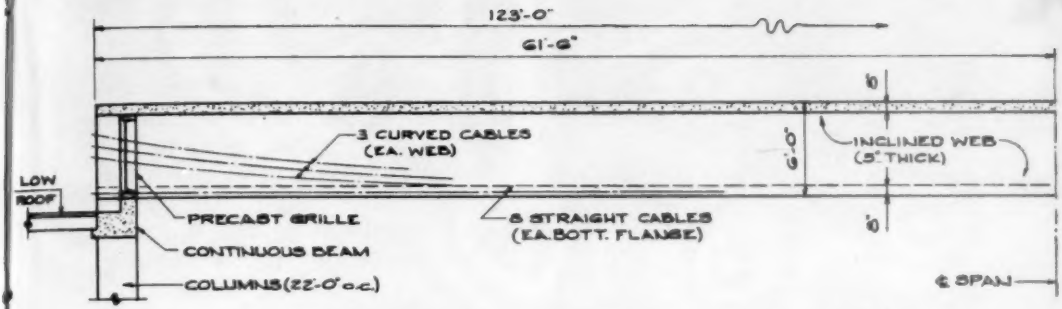


The specified concrete strength of 5000 psi at 5 days with a slump of between 2 and 3 in. was obtained by means of Type III cement and a retarder. Double face forming was specified to insure that the concrete could be properly vibrated without displacing it down the slope.

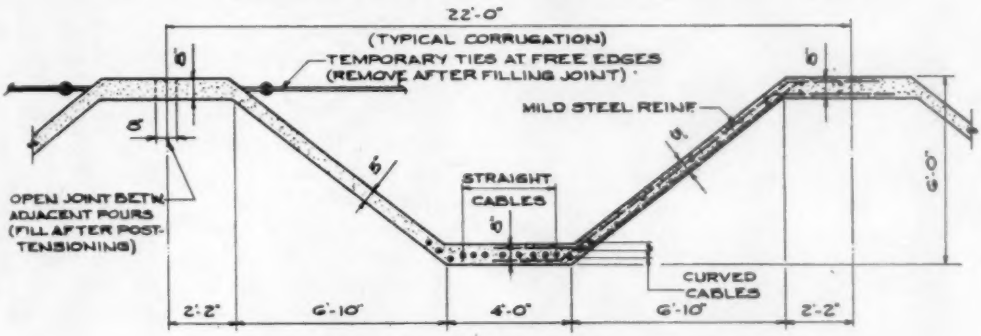
At the start of the roof construction, four corrugations were formed and two were poured. During the time required for this pour to reach the strength required for post-tensioning, the second pair of corrugations was poured. The first pair was then tensioned, stripped, and forms were removed and set up for the third pair of corrugations, while the second pair was attaining tensioning strength. This procedure was followed throughout the roof construction with the exception that a third set of forms for two corrugations was obtained and used. However, by the time they were available, the job was progressing so rapidly that it is doubtful that they appreciably speeded the work.

The soffit forms were supported on prefabricated wood trusses. These were stripped and reassembled for each pour. Upper face forms were prefabricated in sections and removed as the pour progressed. The time required to strip, move and reassemble a complete section for two corrugations, including placing all steel and cables, was four days. In addition to the main roof described

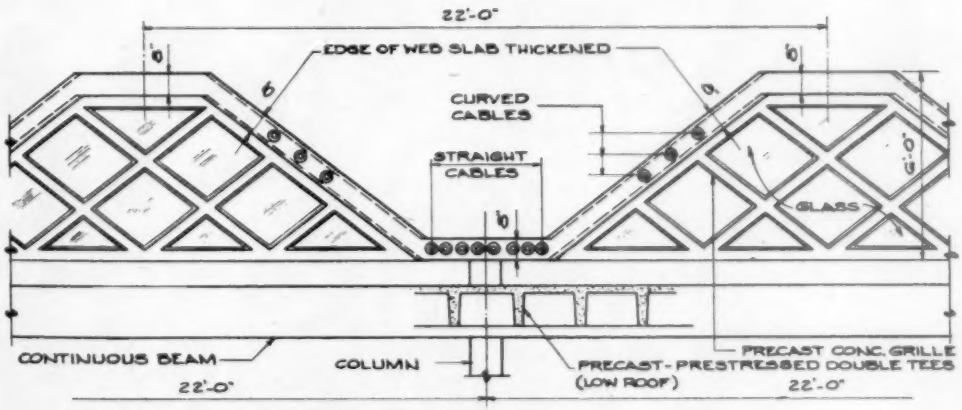




LONGITUDINAL SECTION (HALF SPAN)

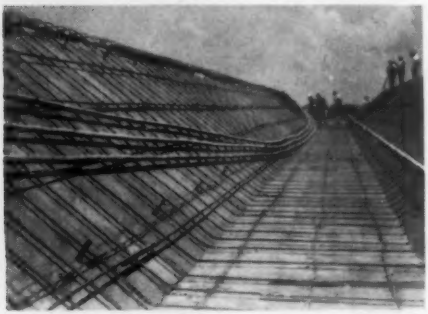


TRANSVERSE SECTION AT MID-SPAN



END ELEVATION (FRONT)

CLOVERLEAF LANES



above, low roof areas also utilized prestressed construction. In these areas, precast, pretensioned double tees were used.

Although the design was not started until February of 1958, it was extremely important to the owner that the building be ready for use by Labor Day of that year. This schedule was complicated by the foundation condition which consisted of very loose sand. Because of the very rigid stability requirements of the alleys themselves, it was necessary to compact the entire building area by means of Vibroflotation.

Vibroflotation began in April. The first pair of roof corrugations was poured on May 15 and the last pair on June 24. The building was ready for bowling on September 1, 1958.

The total cost of the 48,000 sq. ft. building, including air conditioning, electrical, paved parking area, and Vibroflotation, was $439,000. The structural cost was $3.63 per sq. ft.

Alfred Browning Parker of Miami was the architect on the project. General contractor was John B. Orr, Inc., also of Miami. Prestressing by R. H. Wright, Inc., Ft. Lauderdale. Engineered by Dignum Associates, Miami. James P. McGlinchy was in charge of structural design.

Tension in Concrete Piles During Driving

by E. A. L. Smith*

In a recent article published in the *P.C.I. Journal*, Ben C. Gerwick, Jr. discussed the problem of horizontal cracking in prestressed piles during driving and stated that in certain cases this cracking had been eliminated by making the head of the pile circular, so that torsional stresses could not be transmitted to the pile as a result of its engagement with a rectangular or square driving head (helmet). The writer's company has also experienced horizontal cracking in prestressed piles in a number of instances, but in these cases the piles were cylindrical and therefore could not have received torsional forces from the driving head. Investigation disclosed that in these cases the failure was due to excessive tension in the prestressing steel caused by the hammer blow, and the failures were eliminated by using adequate tensile steel and by modifying the driving procedures. The following is an attempt to explain how tension can arise during driving and how its effects may be controlled.

Anyone who has lined up a number of billiard balls in the middle of a table, with the balls touching one another, and has struck the first ball with the cue ball, knows that the last ball will shoot off by itself with a velocity approximately that of the cue ball. If the balls were tied together in some way, the last ball would be restrained from shooting off by itself, but the tying element would have to exert force and would be subjected to tension. In a prestressed pile, the concrete corresponds to the row of balls and the prestressing steel corresponds to the tying element mentioned above, since the concrete has little tensile strength.

But if instead of the row of balls being placed near the middle of the table, they had been so placed that the last ball was against one of the cushions, then the last ball could not have shot off by itself and the tying element would have been subjected to little tension. We may conclude therefore, that objectionable tension in piles occurs only under certain conditions. Let us examine the matter to determine in a general way what these conditions are.

The billiard balls indicate clearly that tension (or separation) occurs when the end ball is free to shoot ahead. The same is true of a pile. If the point of a pile is in soft material, it is free to shoot ahead under the hammer blow and thus cause tension in the pile. If on the other hand the point of the pile has penetrated to hard material, it is not free to shoot ahead and cannot produce tension in the pile.

The point of a pile is almost always in soft material when the first hammer blow is struck. This is when tension is most likely to occur. Sometimes soft material is encountered lower down. The writer's company has found it necessary on numerous occasions to reduce the stroke of the hammer so as to hit a comparatively gentle blow until firm bearing material is reached, after which the hammer can be allowed to hit its full blow. Reducing the stroke

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of the hammer in this way reduces the tensile stresses in the pile and eliminates horizontal tensile cracks. Increasing the amount of prestressing steel used would also tend to eliminate tensile cracks.

In the year 1957 the writer's company made some tests by freely suspending an 80 ft. prestressed pile in a horizontal position and striking it at one end with a 15,000 lb. ram which was allowed to swing like a pendulum so as to hit a blow in a horizontal direction equivalent to

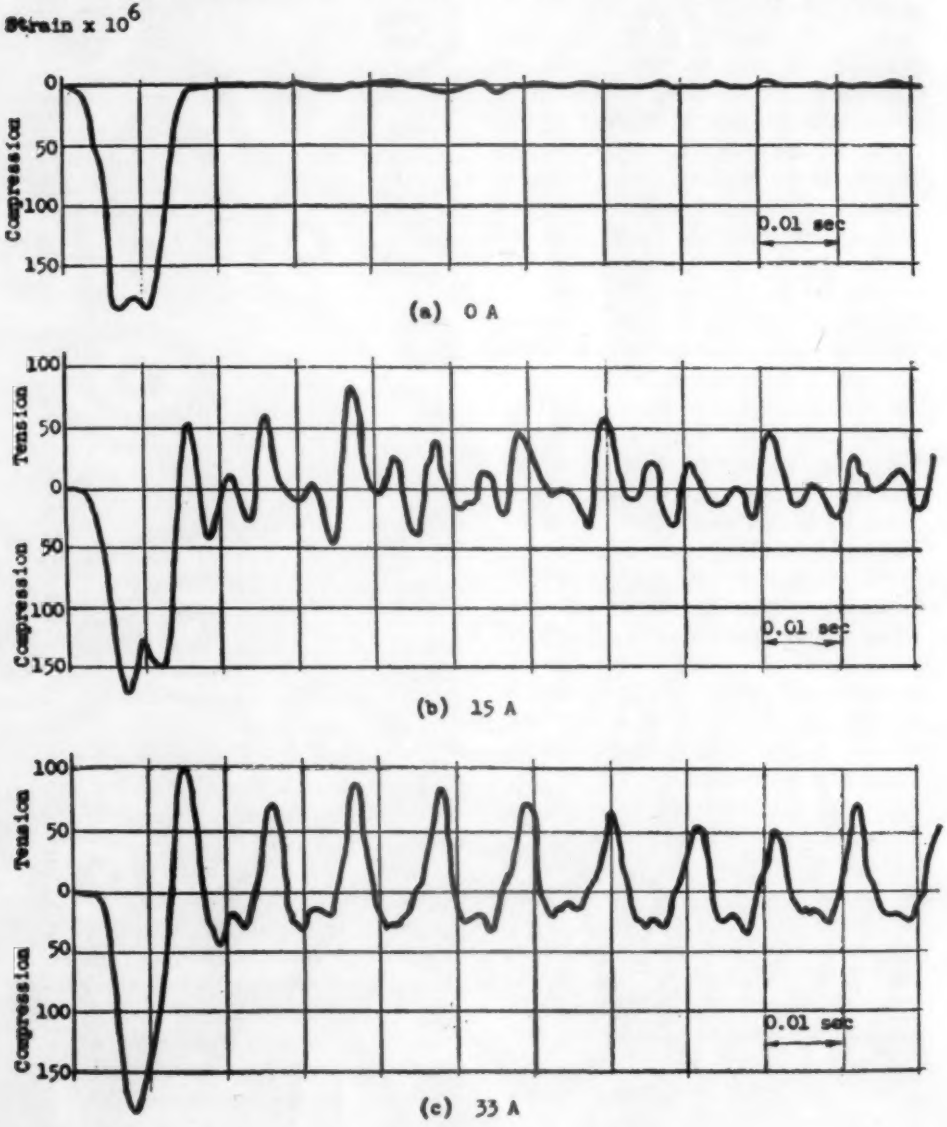


FIG. 1

TYPICAL DYNAMIC STRAIN MEASUREMENTS

(RUN 111 - BLOW 3)

the ordinary blow that would result from a 39 in. vertical drop. The pile was 54 in. in outside diameter with a 4⅜ in. wall thickness. The end opposite the hammer was free, thus the conditions approximated those encountered in actual driving when the point of a pile is in very soft material. The principal measurements made in these tests were strains on the outer surface of the concrete. These strains were measured by means of Baldwin Type A-3 electric resistance strain gauges, having a gauge length of 1 in., attached to the concrete surface with Duco cement. The strains were recorded by oscillographs, and Fig. 1 shows the results of one particular hammer blow. The curve marked (a) was for the head of the pile (b) for 15 ft. from the head, and (c) for 33 ft. from the head. The maximum tensile strain recorded was 100 microinches (millionths of an inch) per inch at 33 ft. from the head of the pile. Assuming a modulus of elasticity of 6,000,000 for the concrete (which was very strong) the stress per square inch in the concrete may be obtained by multiplying by 6, thus the stress corresponding to 100 microinches per inch is 600 psi. The concrete in this pile had been prestressed to about 1,000 psi, therefore no tensile cracking actually occurred during the test.

The mathematics of pile driving is complicated, but making some greatly simplified assumptions for purposes of illustration we may explain how tension can be produced as follows:

Stress waves in a solid material, such as concrete, travel longitudinally with a velocity given by the formula

$$V = \sqrt{\frac{E}{\rho}}$$

where E is the modulus of elasticity, and ρ is the mass per unit volume. Assuming concrete with E = 6,000,000 psi, the computed value of V is 13,600 fps.

Assume that Fig. 2 represents a pile of indefinite length, that has been struck by a hammer. The hammer is assumed to have produced in

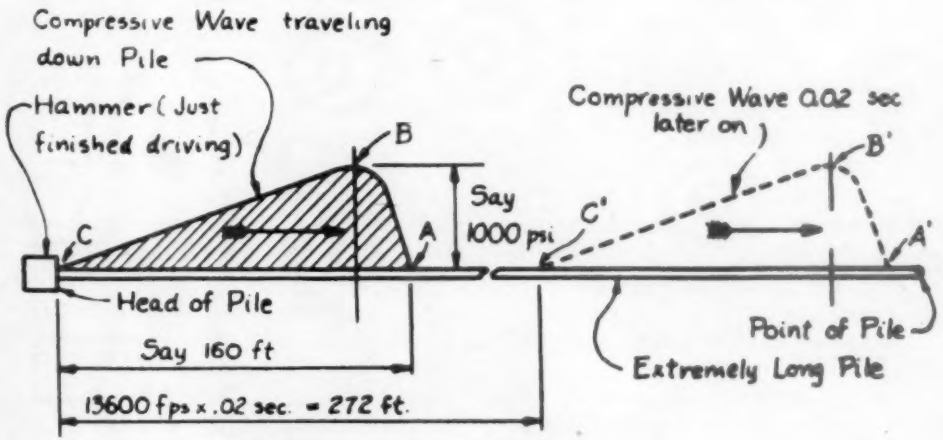


Fig. 2 - Compressive Wave Traveling Down Long Pile.

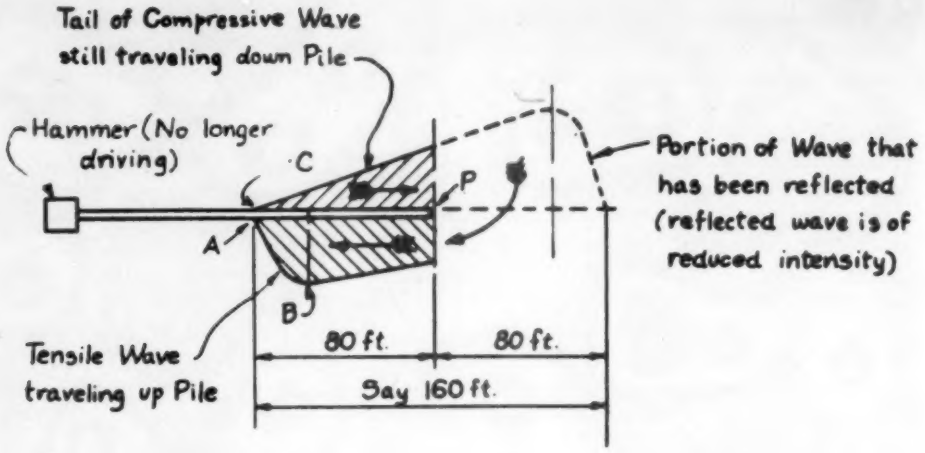


Fig. 3 – Wave Reflection at Point of 160 ft. Pile in Soft Ground.

the pile a compressive wave ABC with a peak stress of 1000 psi and an overall length of 160 ft. When the hammer first struck, the compression began to build up as at A, rapidly reached a peak as at B, then died away and returned to zero stress, as at C, as the ram slowed up and came to rest.

As stated previously this compressive stress wave travels down the pile at approximately 13,600 fps; for instance in Fig. 2 after 0.02 second the wave would be at A′B′C′ which is 13600 x .02 = 272 ft. further down the pile. At any given instant of time there is stress in the pile only where the wave is at that particular instant. Theoretically, if the pile were of infinite length, the wave would travel onward forever, but at any given instant only 160 lineal feet of the pile would be in compression.

If the compressive wave of Figure 2 is applied to a pile of definite length, and if the point of the pile is a "free end" (i.e. if it is in very soft material) the compressive wave caused by the hammer blow will be reflected at the point of the pile as a tension wave of slightly reduced intensity. Considering now a pile 160 ft. long and also considering the exact instant of time when half the length of the wave has been reflected, we get the diagram of Figure 3.

At P in Fig. 3 (the point of the pile) the tensile wave almost balances the compressive wave and there is little stress. Similarly, between the head of the pile and location A there is no stress. However, between A and P the tensile wave is of generally higher stress than the compressive wave, therefore net tension exists. Near B the net tension is maximum. If we wait another fraction of a second, the diagram of Fig. 4 will apply. The reflected tension wave has now progressed up to the head of the pile. The compressive wave has disappeared. The full length of the pile is in tension, and in the vicinity of B the tension is maximum. This tension may be more than the prestressing steel can safely withstand. Horizontal cracks may therefore appear anywhere in the

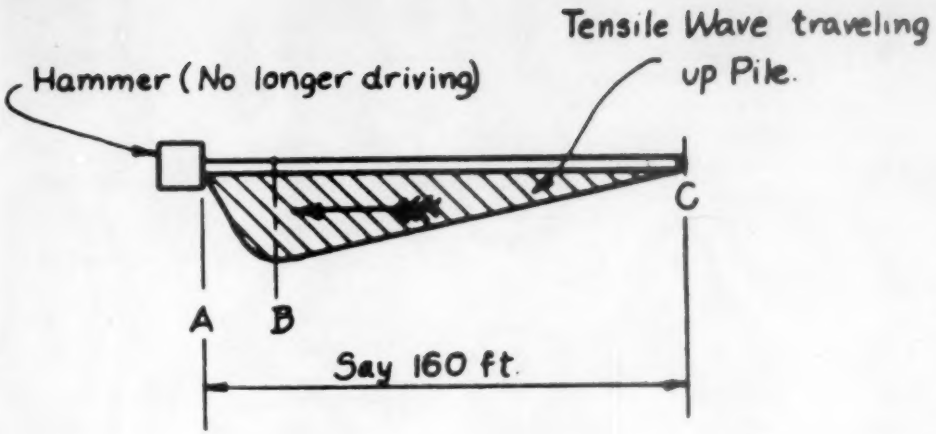


Fig. 4 – Complete Wave Reflection for 160 ft. Pile in Soft Ground.

upper part of a long pile such as this.

If the pile is short, say 50 ft. long, there is small chance of horizontal cracking due to tension. This is illustrated in Figure 5. In this case the peak of the reflected tension wave B is always overlapped by a compressive wave as at D, because the wave length (assumed to be 160 ft.) is greater than twice the length of the pile. Consequently the maximum

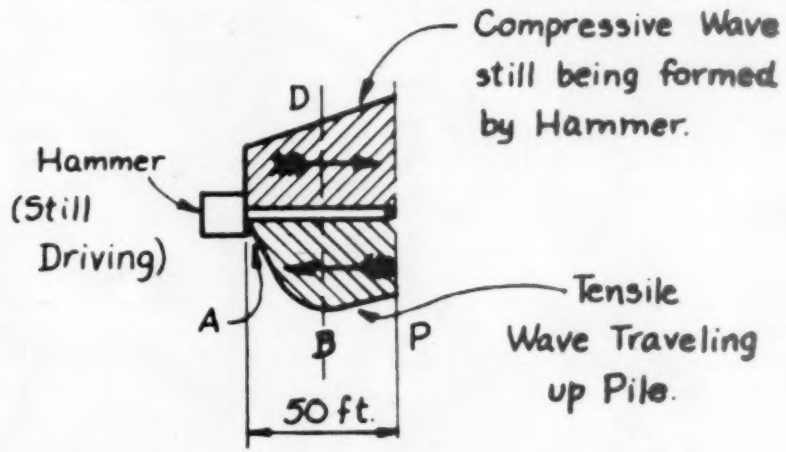


Fig. 5 – Wave Formation and Reflection in 50 ft. Pile in Soft Ground.

net tensile stress in a short pile such as this is not large enough to crack the pile unless the prestressing steel is extra small in quantity or the hammer blow has been unusually severe.

When the point of a pile reaches hard material, the wave caused by the hammer blow is reflected from the point of the pile as a compressive (not tensile) wave. At this time a full hammer blow can be delivered without fear of producing tensile failure. Of course, too severe a hammer blow may cause compressive failure, but that is not the subject of this paper.

From the above the following conclusions may be drawn:

CONCLUSIONS:

1. Prestressed piles are sometimes subject to tension during driving.
2. Long Piles are more likely than short piles to develop horizontal tensile cracks.
3. Horizontal tensile cracks usually appear in the upper half of the pile.
4. Horizontal tensile cracks may be reduced or eliminated by increasing the prestress or by gentle driving using short hammer strokes until such time as the pile point reaches firm resistance, after which full blows may be struck.

Connections By Bond

by

Harry Edwards, President
Leap Associates, Inc.
Lakeland, Florida

The writer discusses various adhesive bonding agents. He believes prestressed concrete will make more extensive use of new bonding materials than any other structural industry.

The history of acceptance of all structural materials has shown a struggle for the discovery and application of a suitable connection for each material. The prestress industry will do well to review the connection evolution in other materials. It will then realize it must go through a similar connection evolution period itself before it can receive its highest acceptance.

Wood, one of the oldest structural materials, is being given a new lease on life and prominence because of improved connections. The split rail fence, the log cabin, the wooden peg, have been with us since man has had timber cutting tools. The nail, while with us only a short time in history, is giving way to inventions in metal connectors and to *glue*. The ability to glue two pieces of wood together with a reliable bond at a low cost has given us plywood and the laminated beam. If the lumber industry had not solved its connection problems, it would be obsolete today as a structural material.

Structural Steel. The structural steel industry has had a 100-year struggle to gain acceptance. The steel industry has given the world a strong material that could be made reliable and consistent in its quality; however, the steel connection problem is still undergoing changes. The riveting hammer is seldom heard. In its place are welding and high strength bolts. However, the steel industry is yet to go through another and probably its greatest stage—bonding by adhesives.

Aluminum. The aluminum industry like wood and steel has been held back by connection problems. It, too, has used bolting, welding and bonding. The latest bonding techniques have been so strong and reliable that they are used in the assembly of fighter plane wings for the elimination of rivets. The new adhesives will open up greater fields for structural aluminum.

Concrete. Precast concrete, too, has solved some connection problems. In the composite field pour and welding have both reached a stage where they are doing a satisfactory job. However, we should look on this stage as comparable to riveting, bolting and welding in structural steel. None of these stages of development are static. Each industry must make its own advancement.

Bonding. Prestressed concrete will probably make the greatest use of all structural industries of the new bonding materials. It needs the simple solution of bonding. It is a growth industry not yet tied down by rigid thinking of too much established know-how. Bonding will permit the designer to go to new and advanced designs of built-up and laminated sections for greater efficiency, reduced weight, and lower costs.

Epoxy. There are many bonding agents, so many that it is confusing. However, at the present stage, there is one that seems to be the glamour boy. It is the epoxy. Being the star, it is also temperamental and costly. It is toxic. It is sensitive to humidity changes during application. It must be applied and cured in a definite temperature range. After mixing with its hardener, it has a short pot life. With all these problems, some might say, "let's forget it." But we believe the rewards gained as a result of more research will be well worth the effort. An epoxy or one of its cousins is probably prestressed concrete's answer to connection problems. Let us see what it might do.

The liquid epoxy must be mixed with a hardener, or so-called catalyst, before its molecules will join hands and set up to be a very firm, tough and strong plastic. During this period, while it is setting, it will bond very strongly to concrete or steel. It has strengths in compression, tension and shear greater than the strongest portland cement concretes, up to and beyond 10,000 psi. Its pot life, or setting time, may be varied from five minutes up to several hours depending on the amount of hardener added and the temperature.

Aggregate or fillers may be added to the epoxy to give it body for use on joining irregular surfaces such as rough concrete castings. These fillers may be silica flour, ordinary sand used in concrete aggregates, or even rock and gravel.

Epoxies may also be applied to steel or concrete surfaces to bond a composite pour of reinforced concrete to this surface. The epoxies used as concrete adhesives have been greatly improved by the addition of polysulfide liquid polymer, commonly known as "Thiokol." Test specimens bonded with "Thiokol"-epoxy adhesives generally fail in the concrete area away from the adhesive bond, indicating that the bond is stronger than the concrete.

Results of Tests

The Thiokol Chemical Corporation has conducted a series of tests of concrete-to-concrete bonding with various plastic formulations. These tests would indicate that shear strengths of concrete with reinforcing steel extending all the way to the glue line will go as high as 4,000 psi and that the failure will not be in the bond of the adhesive, but in the concrete itself. The tests would indicate also that it is feasible to develop full horizontal shear resistance for built-up sections whereby various pieces of precast and prestressed concrete are bonded together to form a structural member. The only disconcerting factor on the bond strength of "Thiokol"-epoxy adhesives are that the strengths drop at elevated temperatures. At 350 deg. F., the shear strength would appear to be 110 psi.

The perfection of a satisfactory bonding technique will permit the industry to go to designs with built-up sections. For example, it would be possible to produce a prestressed

concrete truss out of several separate and individual members. The concrete truss will consist of a top and bottom chord plus diagonals, all of which will be bonded together with adhesives. This assembly and bonding can be done either at the plant or at the job site. It may also be possible to produce very economical Tee sections. These could be produced from flat plates requiring a very minimum of forming expense and a minimum of casting yard labor. Both the stems and the top flange would be made in the flat position directly against the surface of a prestressing slab. After they have been cast, cured and prestressed, they would then be assembled into their Tee sections. It would also be possible to put diaphragms into these thin Tees as required. Such a Tee joist or Tee beam need be no more than 2 1/2 in. in thickness, yet it could be made in variable depth of sections so as to have a span range anywhere from 20 ft. on up through 70 or 80 ft. in span.

Bonded Cover Planks

The use of a bonding adhesive also opens up the way to use a concrete cover plank similar to what is now being used in the steel industry whenever a cover plate is welded to a rolled steel section to increase the strength of that section. An example of the cover plank might be the use of a prestressed concrete plank which would be bonded to the underside of the Double Tee. This concrete plank would be made 2 1/2 in. thick, the width of the Double Tee stem, center distance, and the length of somewhere between 1/3 span and 1/2 span. This plank would be bonded to the underside of the Double Tee covering the center half or center third of the span. The plank would carry heavy prestressing steel. It would be bonded to the stems of the Double Tee for the entire length of the plank. It would be designed so as to take the maximum horizontal shear stresses required which would be zero at midspan and would increase to a maximum at endspan.

Advantages of Bonding

Similar cover plates or cover planks could be also used on Tee sections to convert part of their span from a Tee section to an I section. This procedure will increase the loading capacities of standard prestressed Tees and Double Tees. It will reduce their deflection and will enable them to be used on designs of considerably longer spans. It may also be possible to produce columns, electric light poles and power poles from thin built-up prestressed concrete sections. For example, a building column might be built up of four concrete planks which would be 2 1/2 in. by 16 in. in cross section for each plank. This would produce a hollow core column or pole for long column lengths and high loadings.

The plastic bonding agents can also serve as means for attaching light joists to their supports to resist against uplift. This procedure has already been used on several installations with relatively good success. The bonding agents will also serve well in the future for the attachment of metal hangers to exposed concrete. It will be possible to attach these metal hangers for either light or heavy loads at any position in the building at less effort and less expense than would be required for welding a hanger onto a steel structure.

Prestressed concrete stands to gain considerably from the full de-

velopment of bonding agents used for connections. It will open up an entirely new development program of built-up and assembled concrete members for greater efficiency, lower costs, lighter structures and longer spans. The use of adhesives in the prestressed concrete field can be of much importance as it has been to the timber industry in the development of plywood and laminated beams.

Corrosion Tests on Prestressed Concrete Wire and Strand

Howard J. Godfrey

Assistant Chief Engineer, Research & Development
John A. Roebling's Sons Division, C.F.&I.

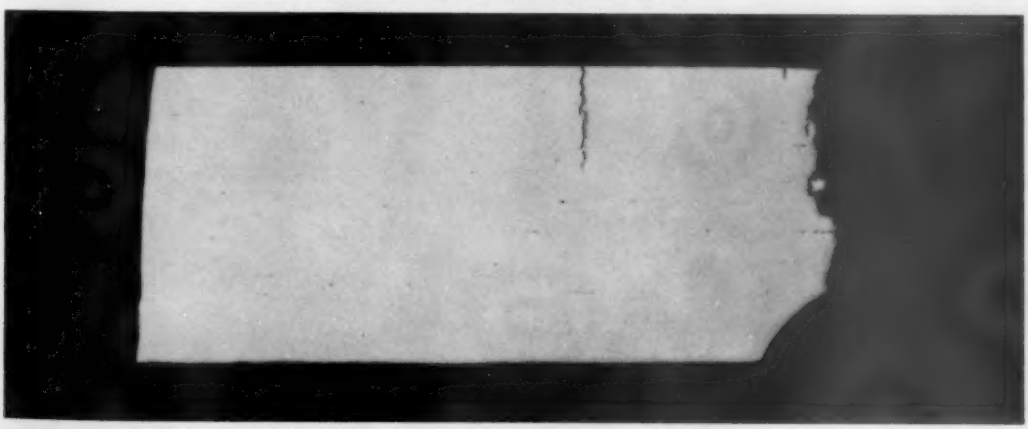
There have been very few corrosion problems in prestressed concrete construction in the United States since the first large structure was built, approximately ten years ago. Under normal working conditions corrosion of the steel does not take place to any serious degree. Experience and laboratory investigations, however, have shown that under certain conditions corrosion can take place and cause severe damage to the steel tensioning elements. This paper is presented to illustrate some of the conditions which should be avoided in order to prevent corrosion, both before and after the steel has been placed in the concrete.

The design stresses for the tensioning elements in prestressed concrete are relatively high compared to allowable stresses in other types of structures, and a high quality product is essential. A large amount of effort has been devoted to the development of a satisfactory steel for prestressed concrete and its performance to date has been very reliable.

The corrosion of prestressed concrete steels has been investigated by other authors[1,2,3] and we are presenting additional evidence to emphasize the need for recognizing specific conditions which would result in a corrosion failure of the steel. These above authors have shown decisively that when a highly stressed steel wire is subject to certain types of corrosion, failure can take place in a distinct manner, as illustrated by Fig. 1. This type of failure is termed a stress-corrosion failure and is recognized by a penetrating

(*Continued on page 48*)

Fig. 1—Longitudinal section through typical stress-corrosion failure in steel wire.



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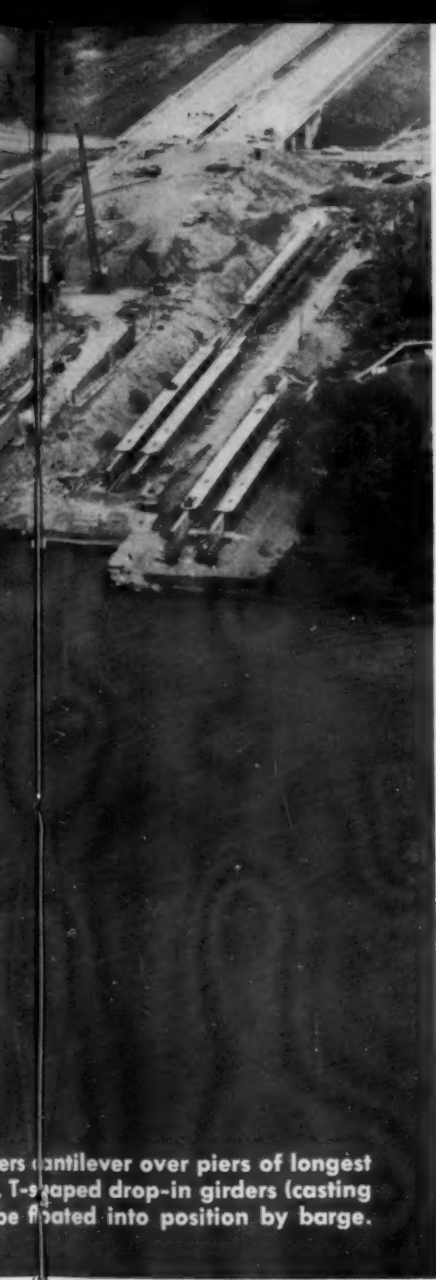

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crack rather than a gradual corrosion of the wire surface.

The previous papers referred to have also shown that oil tempered wire fails much more readily under stress-corrosion conditions than does hard drawn stress-relieved wire. It is for this reason that oil tempered wire is not recommended for use in prestressed concrete. To further indicate the superiority of hard drawn stress-relieved wire over that of oil tempered wire experiments with these two types of wire were conducted in our laboratory.

Type of Wire	Tensile Strength psi.	Applied Stress psi.	Stress Ratio Per Cent
.192 in. dia. Oil Tempered	195,000	117,000	60
.192 in. dia. Hard Drawn Stress-Rel'd.	251,000	151,000	60

The test specimens were stressed in direct tension to 60 per cent of their catalog strength and at the same time placed in a nitrate solution containing 600 gms. per liter of calcium nitrate and 40 gms. per liter of ammonium nitrate. A constant stress was maintained for the duration of the test and the corrosive solution was at room temperature.

The results of the stress-corrosion tests are shown in Table I.

Fig. 2 is a longitudinal section through the oil tempered wire (Spec. No. 2) at the point of fracture, and shows the many transverse cracks which had developed near the point of fracture during the 38 day test period.

Fig. 3 shows a similar section of the hard drawn stress-relieved wire which failed after 650 days. The stress-relieved wire was badly pitted but did not develop any deep penetrating cracks that were so prevalent in the oil tempered wire. The type of rusting and pitting on the stress-relieved wire was similar to that which occurs after long exposure to outdoor weather conditions.

The above tests emphasize the superiority of hard drawn stress-relieved wire and also show that both types of wire should be protected from severe corrosion conditions. It is generally assumed that once the steel is covered with concrete the problem of corrosion can be forgotten. However, our investigational work indicated that certain precautions should be taken in regard to

Table I

Type of Wire	Spec. No.	Time at Failure	Type of Failure
.192 in. dia. Oil Tempered	1	13 Days	Stress-corrosion
.192 in. dia. Oil Tempered	2	38 Days	Stress-corrosion
.192 in. dia. Hard Drawn Stress Relieved	1	650 Days	Rusting and pitting

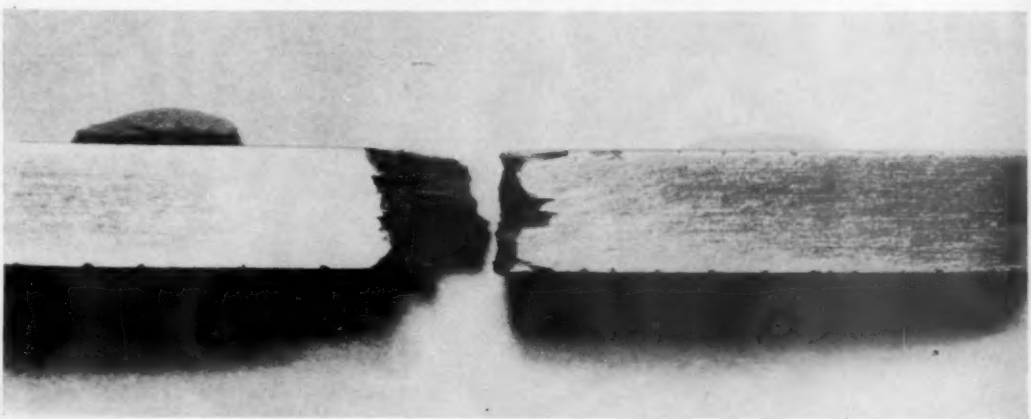
Fig. 2—Longitudinal section through failure in oil tempered wire after 38 days in nitrate solution and under 117,000 psi tensile stress.

the use of admixtures in the concrete mix.

Calcium chloride is used as an additive to concrete mixes to increase the rate of hardening and thus reduce the time at which prestressing can take place. The results of a study on the effect of calcium chloride on the corrosion of steel wire and strand are described herewith. The concrete mix contained 7.5 bags of cement per cubic yard of concrete and 5 gallons of water per bag of cement. One series of beams was made without calcium chloride and a second set of beams was made with 2 per cent calcium chloride (2 per cent of the cement content). The strength of the concrete as determined from 2 in. cubes at the age of 6 months was 6,000 psi.

The beams used for the tests were 3 in. x 3 in. x 48 in. and contained one 3/8 in. 7-wire prestressed concrete strand placed in the center of the cross section. The steel strand

Fig. 3—Longitudinal section through failure of hard-drawn stress-relieved wire after 650 days in nitrate solution and under 151,000 psi tensile stress.



was pretensioned to 14,000 lbs. or 175,000 psi. prior to pouring the concrete.

The beams were exposed in an industrial atmosphere on the laboratory roof for a 3-year period, at which time one set of each type of beam was broken up and the steel examined. As illustrated in Fig. 4, the steel strands in the beams containing no calcium chloride were free from rust or any other signs of corrosion. The strands in the beams containing calcium chloride were rusted and the wires were severely pitted. The corroded condition of these strands is also illustrated in Fig. 4. The strands were corroded along the entire length of the 4-ft. beams and most of the corrosion took place on the underside of the strands. Corrosion was observed on the center wire of the strands as well as on the six outside wires. The strands taken from both sets of beams were tested for their tensile strength and the results are presented in Table II.

The tensile tests on the 3/8 in. diameter strand from the beams without calcium chloride showed no loss in strength or elongation from the original material.

The strands from the beams containing calcium chloride showed a 5 per cent loss in tensile strength and a 60 per cent loss in elongation.

The above tests indicate that prestressed concrete beams without calcium chloride in the mix were in a very satisfactory condition after 3 years exposure in an industrial atmosphere. The steel strands in the beams containing calcium chloride, however, were badly corroded and indicate that calcium chloride should not be permitted in prestressed concrete structures.

A recent article by Baumel and Engell[4] confirms our results on the corrosion of steel in concrete containing calcium chloride.

Prestressed concrete has proven to be a very satisfactory and economical type of construction in this country. Every effort should be made to maintain this position by using proper construction methods. Protection of the steel from corrosion is essential for continued success in prestressed concrete construction.

Dr. Fritz Schwier—Wire and Wire Products, December 1955
W. O. Everling—Journal American Iron & Steel Institute, 1954
G. T. Spare—Wire and Wire Products, December 1954
A. Baümel and H. J. Engell—Archiv für das Eisenhuttenwesen, July 1959

Table II

Tensile Tests on 3/8 in. Dia. Prestressed Concrete Strands After Three Years in Concrete Beams

Spec. No.	Type of Concrete	Breaking Strength-lbs.	Per Cent Elongation in 24 in.
1-A-1	Without Calcium Chloride	22,550	6.7
2-A-1	Without Calcium Chloride	22,650	*
1-B-1	With Calcium Chloride	21,350	2.6
2-B-1	With Calcium Chloride	21,500	*

*Elongation not determined.

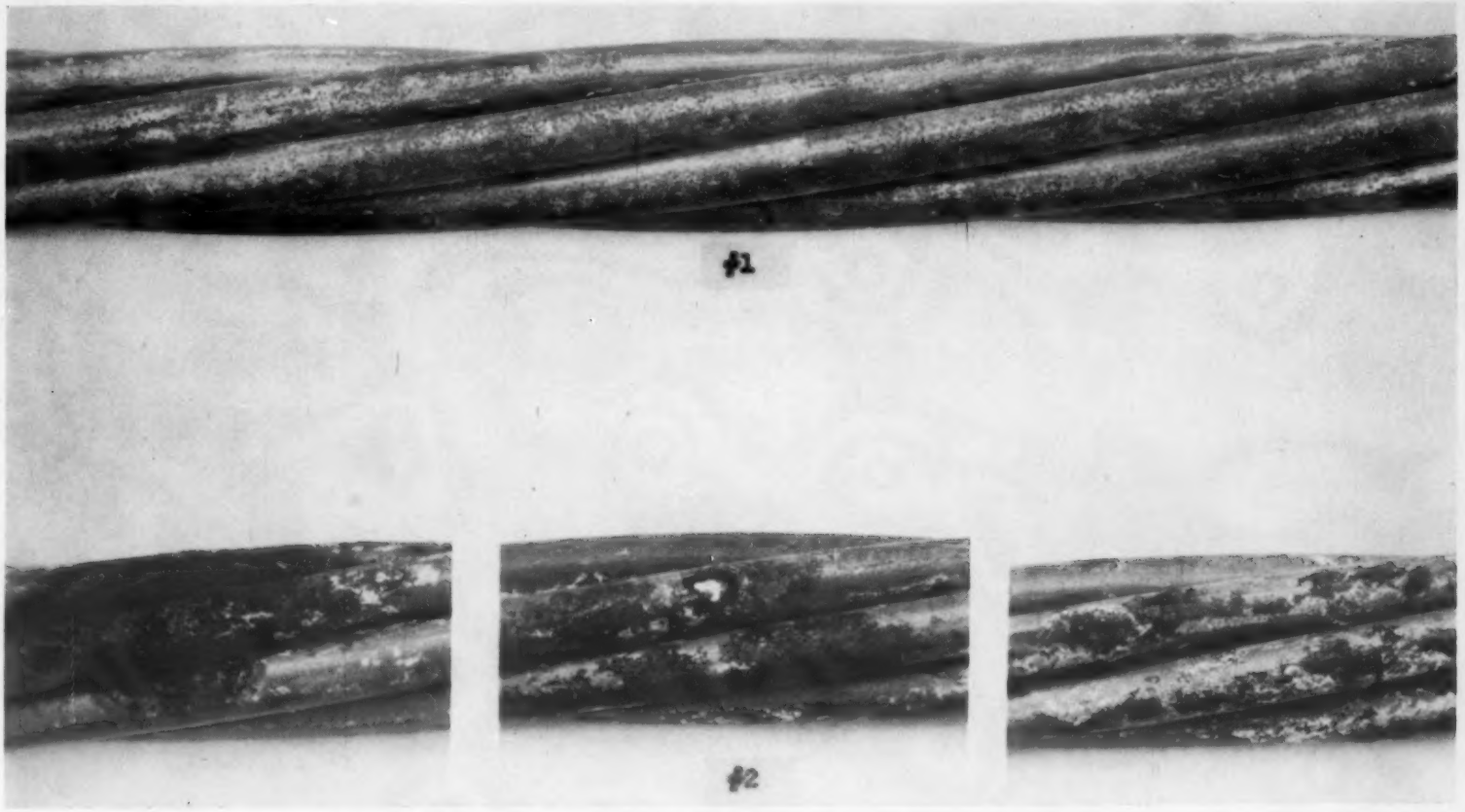


Fig. 4—Surface view of 3/8 in. dia. prestressed concrete strands removed from concrete beams after 3 years exposure in an industrial atmosphere.
No. 1 is concrete with no calcium chloride.
No. 2 is concrete with 2 per cent calcium chloride.

The Montrose Avenue Bridge

over the Edens Highway and Northwest Expressway Cook County, Illinois

By M. Schupack,

Partner, Schupack & Zollman, Stamford, Conn.

SYNOPSIS

A three span cast-in-place prestressed concrete bridge with 154-ft. spans was constructed in Cook County in 1958. This bridge again indicates that prestressing concepts can be practically and economically applied to the larger span bridge field.

The construction of longer span prestressed concrete bridges is still in the earliest stage of development in the United States. This is in contrast to the European practice where longer span prestressed bridges of 100 to 300 ft. plus have virtually displaced the use of structural steel bridges.

Of the dozen or so prestressed bridges in the United States with spans over 120 ft., the Montrose Avenue Bridge is another indication of the potentials and possibilities in this still virtually untapped market in the prestressed concrete field. This bridge was bid on November 9, 1957. The bid tabulation below shows the favorable prices received. The cost of the bare superstructure amounts to $13.53 which is a good competitive price for a bridge deck of 154 ft. spans and a 7 ft. construction depth limitation.

The bridge, in Chicago's northside, consists of three simple spans of 154 ft. each carrying Montrose Avenue at a 40 degree skew over the junction of the Edens and Northwest Expressways. The 81 ft. bridge width out-to-out of parapets is made up of two 30 ft. roadways, two 8.5 ft. sidewalks and a 4 ft. median. The bridge is divided in half by a joint in the median.

BID PRICES FOR SUPERSTRUCTURE

Item	Unit	Quantity	Bid Price	Total
Cast-in-place Concrete (5000 psi)	c.y.	2774	$102.50	$284,300
Sidewalk, parapet and mall concrete	c.y.	110	51.00	5,600
Prestressing steel	lump sum			133,000
Reinforcing steel	lbs.	347,000	0.15	52,100
Structural steel	lbs.	107,000	0.40	42,800
				$517,800

The completed Montrose Ave. Bridge: 3 spans at 154 ft.; cast-in-place monolithic "T" beam design.

The choice of construction approach seemed to favor a cast-in-place solution because of the long span and the ease supplying falsework. In actuality, the contractor elected to cast the deck on grade, using temporary concrete soffits for support. Continuity was considered, but not used for the following reasons:

a) Did not permit as much reuse of falsework and formwork as simple span construction.

b) Did not permit construction of substructure and superstructure in an almost simultaneous operation.

c) Since three equal spans were dictated there was no particular economy of material when compared to simple spans.

d) Prestressing details became unduly complex for this particular job.

e) The saving in bridge shoes, and expansion joints did not seem to effect the expeditious construction approach possible with simple spans.

Selection of Cross-Section

The piers consisted of individual shafts, spaced approximately 13 ft. on center in the skew, founded on spread footings. This type of pier was chosen for simplicity and economy. One pier shaft was placed under each web which was spaced 8.5 ft. on center, with the result that the spacing of pier shafts was dictated by the spacing of webs in the cast-in-place superstructure. It would have been preferable to use a larger spacing between pier shafts. However, the detail problem of finding sufficient room for locating the commercially available prestressing tendons in fewer webs, precluded wider web spacing. The webs were tapered

in lieu of using rectangular webs in order to economize on prestressing requirements. If a rectangular web was used more prestressing force would have been required, thus making it difficult to place the tendons in the webs and thus requiring an additional web.

The prestressing system selected by the contractor and approved by the consultants was the BBRV system. The individual units consisted of 40-0.250 in. diameter wire. For longer spans, the use of more massive tendons has many advantages. This tendon had an initial force of approximately 320 kips. If the anchorage and stressing details remain practical, even larger capacity tendons could be used.

The actual construction of this bridge had no particular problem except in getting the subcontractor to appreciate the requirements of placing tendons. This was rapidly mastered after the first span. The placing of concrete was done most expeditiously. It took a crew of 14 men with two cranes, using transit mix, approximately nine hours to place 460 cu. yd. of concrete. The concrete was gravel aggregate with maximum 1 in. stones and contained seven sacks of cement per yard and 6 ounces of a retarder per sack. In weather with temperatures averaging 60 deg. F., 4500 psi concrete was obtained in twelve days and well over 5000 psi in twenty-eight days.

The use of the existing grade as the support for the construction of the superstructure required careful

Bridge was built on existing ground. Later soil was excavated out from under. Note preparation, in foreground, for beam soffits.

Placing concrete on the first half-span.

excavation of the soil after the prestressing was introduced. Some friction of the soffit forms against the elastic shortening of the concrete was encountered when prestressing. However, this did not create any problems.

The stressing of the tendons was done expeditiously after the contractor became familiar with the equipment. All tendons were stressed from one end only and the expected friction for this stressing procedure was reflected in the design. The actual friction was less than calculated and adjustment to stressing was made accordingly in the field. The prestressing of this bridge now accounts for the prestressing of 44 individual spans each more than 120 ft. long, by the means of one end stressing. No trouble of any kind has been encountered in our experience.

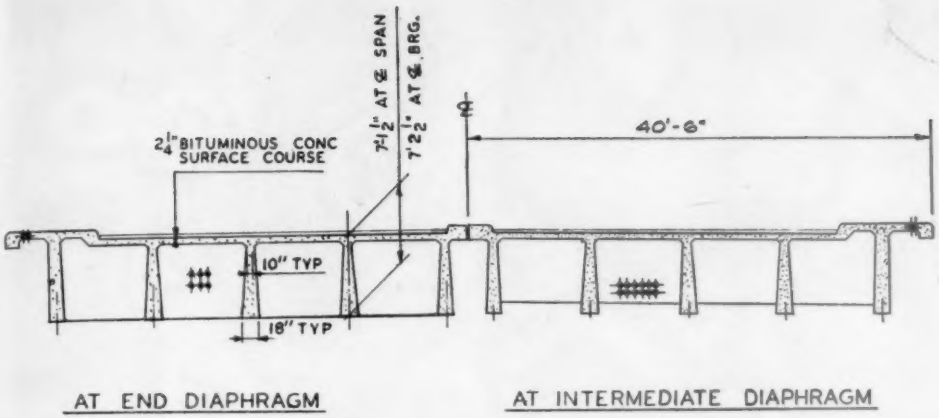
The design of a structure of this type follows all the usual procedures of prestressed concrete theory. In this case the 40 deg. skew actually could have an appreciable affect on the deck action of this bridge if it were a monolith for its entire width. However, a longitudinal joint was used in the median to obtain more individual units of construction and also to avoid transverse temperature problems. The bridge width along the skew line is 106 ft.

The choice of section in bridges of this type, is generally dictated by construction considerations such as reuse of formwork, tendons to be

"Trumpets" to house prestressing anchorages, are bolted to plywood bulkhead. Note prefabricated chairs to support prestressing steel. At right is formwork for interior diaphragm.

placed, sufficient space for concreting and vibrating. Based on our experience with bridges of this type we can generally estimate the approximate amount of prestressing steel required, and determine that a minimum number of web stems will result in a minimum of formwork and web reinforcing setups. Based on the commercially available

After partial prestress, a tunnel was excavated under one end of the beams. The formwork was removed through this tunnel.



TYPICAL SECTION

prestressing tendons and their end anchorage space requirements, the web spacing is determined. It seems to generally follow that the concrete which comfortably covers a tendon layout is close to the theoretical concrete required.

Our design approach is not elegant nor is it direct design, it is really an investigation of a practical section for all the construction requirements. For spans of this range we have found that our experience gives us the most direct approach.

The stress table shown below is the stress condition at midspan. The relatively small change in stress due to live load again demonstrates some of the excellent structural characteristics of prestressed concrete.

Some of the not too well known characteristics of prestressed concrete, particularly in the more massive sections, is the volume change characteristics. We have found that the camber could be predicted within ¾ of an inch for 154 ft. spans. However, our estimate of the rocker

Stress in an Interior Girder Before and After Prestress Losses:

Fiber Stress	(1) P/A	(2) Pe/Z	(3) (1)+(2)	(4) M_{DL}/Z	(5) (3)+(4)	(6) M_{WS}/Z	(7) (5)+(6)	(8) M_{LL}/Z	(9) (7)+(8)
Initial:									
Top	....	....	−1090	+1650	+560	+200	+760	+660	+1420
Bottom	....	....	+5210	−2790	+2420	−330	+2090	−1130	+960
Final:									
Top	+1030	−1920	−890	+1650	+760	+200	+960	+660	+1620
Bottom	+1030	+3250	+4280	−2790	+1490	−330	+1160	−1130	+30

Note: Stresses are in psi. Plus stresses are compression. Nomenclature and data are as follows:

P = prestressing force
M_{WS} = moment (wearing surface) of 2¼-in. asphaltic concrete
M_{DL} = dead-load moment
M_{LL} = live-load moment
Z = section modulus
e = eccentricity
e = 54 − 6 = 48 in.
Pe = 7,480 kip-ft
A = 1,810 sq. in.
Z_T = 47,000 in.^{3}(top)
Z_B = 27,700 in.^{3}(bottom)

After final prestress, the soil was excavated from under the span.

tilt to account for elastic, shrinkage and plastic shortening was over-estimated. It would be to the advantage of the progress of longer span, prestressed concrete bridge development, if the camber and change in length characteristics would be measured and published.

This was a Federal Aid project under the direction of W. J. Mortimer, Superintendent of Highways, Hugo J. Stark, Chief Engineer, and Milton Page, Bridge Engineer of the Cook County Highway Department. The writer was responsible for the design of the entire bridge, and field supervision of the prestressed concrete portions of the structure with Eugene Marlowe of Schupack and Zollman as resident advising engineer. Joseph T. Ryerson & Son of Chicago performed the post-tensioning operations.

Composite Construction for Continuity

By

Eivind Hognestad, Alan H. Mattock and Paul H. Kaar*

(Presented during 1959 PCI Convention)

SYNOPSIS

Past activities of the PCA Research and Development Laboratories in the field of prestressed concrete are reviewed, and the fundamental nature of this early work is emphasized.

The highlights of a recent extensive laboratory investigation of continuous precast-prestressed concrete bridges are presented. The type of bridge under study includes precast I-shaped girders connected into an integral bridge unit by a situ-cast deck slab. Continuity from span to span is obtained in such composite construction by placing deformed bar reinforcement longitudinally in the deck slab across intermediate piers. The investigation is being carried out in nine related stages, culminating with testing of a complete bridge deck over two spans.

Aside from their application to the specific bridge problems involved, the results of these studies of composite construction have important bearing on building construction. Preparations are therefore in progress for similar broad studies of precast-prestressed buildings.

Past Prestressing Work at PCA Laboratories

In mid-June of 1948, ground was broken in Skokie, Illinois, as the first step toward modern research and development facilities for the portland cement industry of the United States and Canada. The scientific institution that came into being during the decade that followed is devoted to all aspects of cement and concrete technology, from deeply basic research to highly practical engineering development.

Most of the work of the Portland Cement Association Laboratories has a direct or indirect bearing on the prestressing industry. For example, work on cement hydration has aided the development of ac-

*Manager, Senior Development Engineer, and Development Engineer, respectively, Structural Development Section, Portland Cement Association Research and Development Laboratories, Skokie, Illinois.

celerated curing procedures for plant production; studies of lightweight aggregate concrete technology have facilitated the development of improved prestressed buildings; and extensive studies of concrete durability have contributed toward long-lasting concrete bridges. Also, the extensive facilities of the new Fire Research Center are being used to develop knowledge regarding the fire resistance of prestressed concrete structures. Our findings are available in the publications seen in Fig. 1.

This brief paper is limited to presenting the highlights of our structural development work having a direct bearing on prestressed concrete structures. Such work began in 1950, soon after the initial facilities of the laboratories were completed, and a fundamental path was followed in the begining. The mechanism of bond action between pretensioned reinforcement and concrete was studied thoroughly[1,2], and design criteria were developed. The flexural performance of prestressed and conventionally reinforced beams was compared[3], with particular attention to strength, deflection and deflection recovery. A great deal of work was devoted to ultimate strength design theory[4,5,6]. The principles of inelastic action and strength developed in this work are equally applicable to conventionally reinforced and prestressed concrete. This is also the case for more recent work on structural concrete design theory, involving redistribution of moments by limit design[7,8,9]. Only one major investigation departed from this path of fundamental study, namely development of a prestressed concrete railroad trestle slab[10].

As part of these studies of fundamental characteristics of structural concrete, investigations were undertaken of various means of connecting

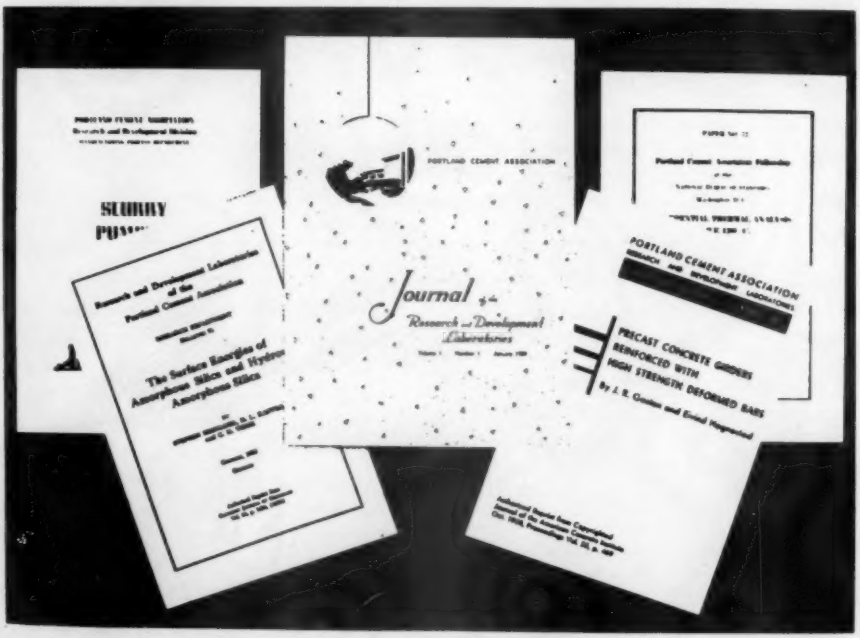

SLURRY
PORTLAND CEMENT ASSOCIATION
Journal of
Research and Development
Laboratories
The Surface Energies of Amorphous Silica and Hydrous Amorphous Silica
PORTLAND CEMENT ASSOCIATION
PRECAST CONCRETE GIRDERS REINFORCED WITH HIGH STRENGTH DEFORMED BARS

Fig. 1—Publications of PCA Laboratories.

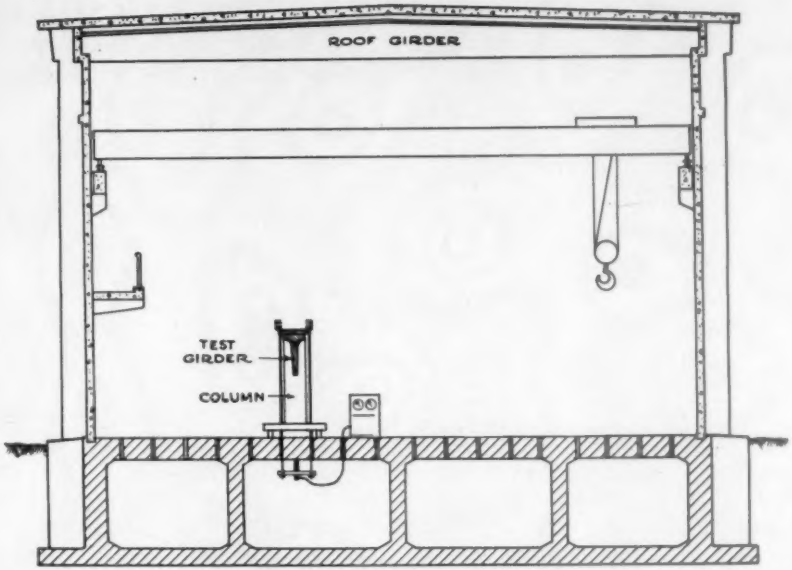


Fig. 2—Cross section of Structural Laboratory.

precast members into integral structures. It became evident that highly effective structural connections could be obtained at low cost by combined usage of precast and situ-cast concrete. Thus, we began laboratory investigations of composite construction.

The work of many investigators at home and abroad led to far-reaching changes and improvements in design theory in the mid-1950's. At the same time, growth of the prestressed concrete industry began to increase rapidly. It became clear that the evolution of structural concrete that began in the 1890's was becoming almost a revolution. It became clear that, to continue progress, large-scale and complex experiments were needed to bridge the gaps between fundamental structural research and the engineering arts of practice, for which experiments no laboratory in the United States was adequately equipped. Thus, it became desirable for the Portland Cement Association to build an unusual new structural laboratory facility.

PCA Structural Laboratory

To accommodate large-scale tests of structural assemblies, rather than individual components only, the PCA Structural Laboratory was built in an unusual manner. Instead of housing conventional testing machines, the laboratory was built as a giant testing machine in itself. As shown in Fig. 2, the laboratory contains a test floor built as a heavy box girder 12 ft. thick and 56 by 121 ft. in plan. Test structures are secured to this floor, and loads are applied by hydraulic rams acting through various arrangements of cross bars and tie rods, as illustrated by the example shown in Fig. 3.

In this manner, the facilities required for thorough studies of composite construction and other complex structural forms became available. The test methods which were developed[11] may also be use-

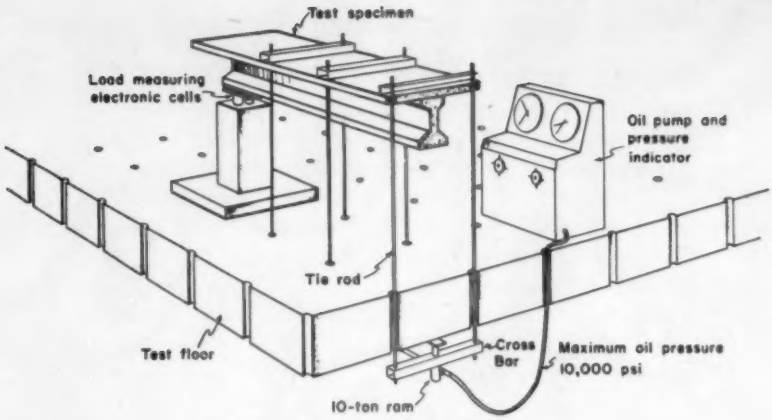


Fig. 3—Hydraulic loading of bridge girder.

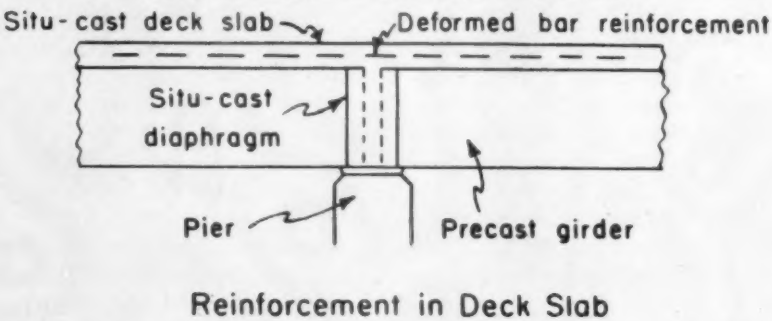
ful in tests at prestressing plants of new prestressed products and the corresponding structural assemblies.

Experimental Investigation of Composite Construction

The fundamental principles of composite bridge construction are illustrated in Fig. 4. A series of precast girders are erected on piers with the girder ends about 6 in. apart. Forms for a deck slab are then erected and supported by the precast girders. In addition to the usual slab reinforcement, deformed reinforcing bars are placed longitudinally in the deck slab over the piers to transfer negative moments from span to span. The transverse diaphragms are then cast, together with the deck slab. In this manner, the situ-cast deck slab and the diaphragms serve a four-fold purpose: (1) providing a roadway accommodating traffic, (2) increasing the strength and stiffness of the precast girders, (3) connecting the girders laterally, and (4) providing continuity from span to span between precast girders. Similar composite construction is used for buildings. The function of the situ-cast bridge slab may then be served by a topping cast on precast channel or double-tee slabs, without the need for additional forms.

This type of composite construction combines precast, situ-cast, prestressed and conventionally rein-

Fig. 4—Continuity connection in composite construction.



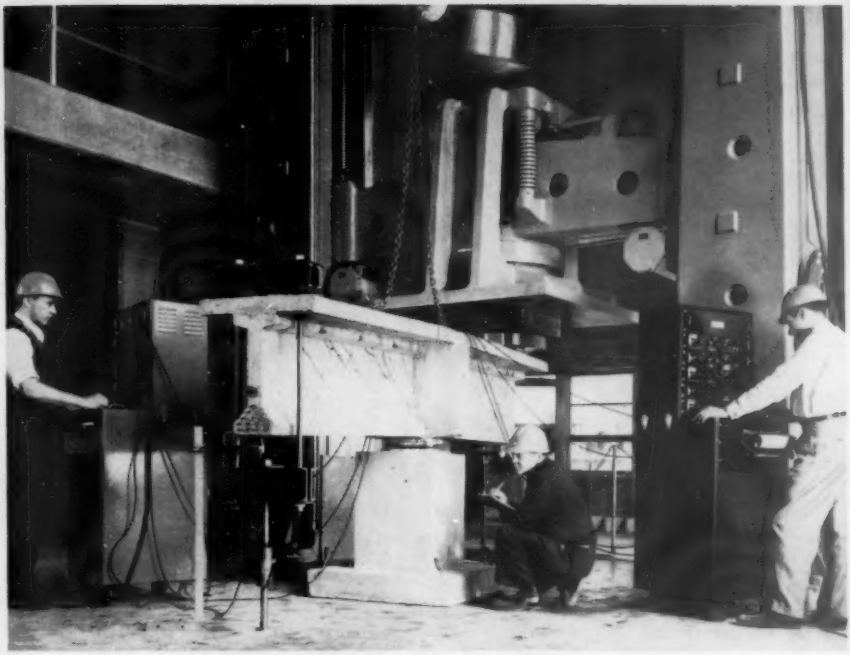


Fig. 5—Static Test of continuity connection.

forced concrete. The mass-production advantages of precasting and the continuity of cast-in-place construction are merged into a single structure. However, composite construction also leads to several questions regarding properties of structural concrete not covered previously by extensive tests. To answer these questions, an experimental investigation of broad scope is now nearing completion at the PCA Laboratories. With specific reference to composite bridge construction, nine related projects were undertaken.

(1) Girder Continuity

The first project of the investigation concerned the performance of the continuity connection under static loads. Tests were made in which the specimens consisted of two precast-prestressed girder stubs joined together by a transverse diaphragm and a reinforced deck slab cast on top of the girder stubs. A typical testing arrangement is shown in Fig. 5. The major variables covered were (1) the amount of precompression in the bottom flange of the prestressed girders, and (2) the amount of deformed bar reinforcement placed in the deck slab to develop continuity. It was found that this type of composite construction for continuity is basically sound, and suitable fundamental design criteria were developed.

(2) Horizontal Shear

The transfer of horizontal shear between precast girders and the situcast deck slab is of vital importance in this type of composite construction. A project was therefore carried out to study the usefulness of various means of horizontal shear trans-

(*Continued on page 66*)

DICKERSON USES LESCHEN

Leschen Strand selected for one of the largest strand patterns tensioned in Pennsylvania

Dickerson Structural Concrete Corporation, Youngwood, Pennsylvania, is producing prestressed concrete box beams for highway bridges in Western Pennsylvania. The Allegheny County bridge shown utilizes beams 88′8″ long by 4′ wide by 3½′ deep which weigh 37 tons each. Design plans called for seventy strands of ⅜″ diameter in a pattern across the base and partially up the sides.

Dickerson is one of an increasing group of *Leschen Prestress Strand* customers who value the sound advice and practicality of Leschen engineers.

The technical requirements of prestressing, too, require unvarying strand quality—where every foot matches every other foot. At Leschen, this high standard is made possible by a new continuous-flow technique in today's most thoroughly modern wire mill. Here new processes, new machines, new methods maintain standards and quality that are above the industry's specifications.

If you would like to talk with a Leschen Strand engineer or receive complete specifications and other helpful information, write today to *Leschen Wire Rope Division, H. K. Porter Company, Inc., St. Louis 12, Mo.*

LESCHEN
PRESTRESS STRAND

Mr. Earle A. Butts, Jr., Dickerson Manager, looks over the strand template used in tensioning the Allegheny County strand patterns. He reports that Leschen Prestress Strand has performed as a well engineered and high quality product.

Leschen strand has helped the Dickerson plant to meet varying design requirements with box beams. The Allegheny County bridge beams were cast four at a time on a 440-foot bed, requiring over 30,000 feet of Leschen 3/8" Strand per pour. Tension: 14,000 pounds per strand or a total force of 980,000 pounds which is converted to a compressive force in the beam.

Positioning these same 37 ton beams on location at highway bridge over Peters Creek on T.R. 51 in Allegheny County near Pittsburgh. These beams with a minimum of construction depth, replaced an existing steel bridge.

LESCHEN WIRE PORTER ROPE DIVISION

H.K. PORTER COMPANY, INC.

PORTER SERVES INDUSTRY: with Rubber and Friction Products—THERMOID DIVISION; Electrical Equipment—DELTA-STAR ELECTRIC DIVISION, NATIONAL ELECTRIC DIVISION; Copper and Alloys—RIVERSIDE-ALLOY METAL DIVISION; Refractories—REFRACTORIES DIVISION; Electric Furnace Steel—CONNORS STEEL DIVISION, VULCAN-KIDD STEEL DIVISION; Fabricated Products—DISSTON DIVISION, FORGE AND FITTINGS DIVISION, LESCHEN WIRE ROPE DIVISION, MOULDINGS DIVISION; and in Canada, Refractories, "Disston" Tools, "Federal" Wires and Cables, "Nepcoduct" Systems—H. K. PORTER COMPANY (CANADA) LTD.

fer. The major variables were: (1) adhesive bond, (2) roughness of the precast girder top surface, (3) shear keys, and (4) stirrups protruding from the precast girder into the situ-cast clab. Two types of specimens were used. In the first type, a short length of deck slab was pushed off a section of precast girder as shown in Fig. 6. Composite T-beams were also tested, for some of which the contact surface between girder and slab was reduced to obtain failures in horizontal shear along the joint. The relationships between slip and shearing stress obtained by the two test methods agreed fairly well. The results indicate that the maximum horizontal shearing stress before breakdown of composite action is about 500 psi for a rough bonded contact surface, and 300 psi for a smooth bonded surface. The results also indicate that the contributions to shearing strength of bond and keys are not additive. In the remaining projects of this investigation, therefore, shear keys were not used.

(3) Bridge Design Studies

In this third project, the tentative bridge structure shown in Fig. 7 was designed for H20-S16 loading, using information obtained in the first two

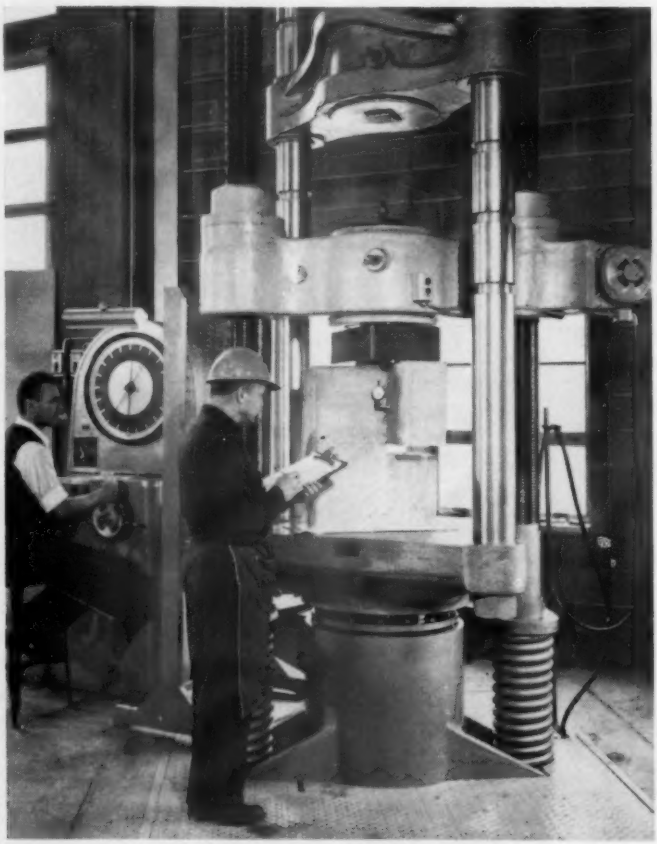


Fig. 6—Horizontal shear test using push-off specimen.

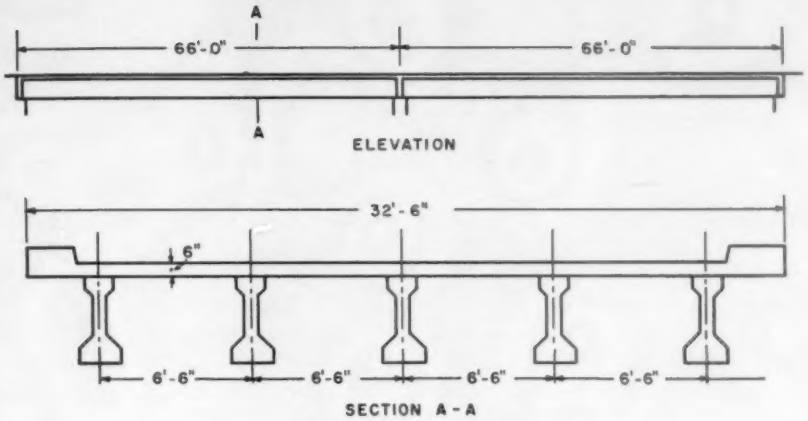


Fig. 7—Composite bridge considered in design study.

projects of this investigation, in addition to design methods already available elsewhere. These design studies brought out a need for additional test data, and further projects of the investigation were then planned.

(4) Diagonal Tension

Near intermediate supports, the continuous composite girders were designed for flexure as conventionally reinforced concrete members. A question therefore arose regarding selection of a proper method of diagonal tension and web reinforcement design. Nominal shearing stresses, calculated on the basis of the web thickness of the precast girders, were unusually high, so that the web reinforcement required by customary reinforced concrete design methods seemed to be excessive. A project was therefore undertaken, involving tests of half-scale composite girders loaded by a group of point loads simulating the wheel loads of the H20-S16 design vehicle. The test girders were supported over a single span with a tied-down cantilever at one end, as seen in Fig. 8. By suitably varying the tie-down force at the end of the cantilever independently from the vehicle loads, conditions were established in the single span corresponding to those existing in one span of a two-span continuous girder. The variables studied were (1) amount of vertical stirrup web reinforcement, and (2) location of the applied loads—that is, the degree of flexure combined with shear. The test results indicate that the presence of prestress in the precast girder has some beneficial effect on shearing strength even in regions of negative bending moment.

(5) Flexural Strength

To extend the findings of the first project to the particular conditions involved in the bridge structure shown in Fig. 7, tests were made of a half-scale single girder continuous over two spans. As shown in Fig. 9, a loading was selected to simulate the equivalent lane loading of H20-S16. The loads were arranged to give the most severe bending moment conditions at the intermediate support. The composite continuity

connection behaved in an entirely satisfactory manner. Full redistribution of bending moments took place before the ultimate strength was reached.

(6) Creep and Shrinkage Studies

Two half-scale 66-ft. continuous girders were placed under a sustained loading simulating dead load of the full-scale prototype 132 ft. long, as shown by Fig. 10. This was done to study effects of creep and differential shrinkage on the continuity behavior of the composite structure. Extensive measurements were made of strains, deflections and support reactions. It was found that, to ensure fully continuous behavior at service loads after an extended period of time, small reverse moments set up at interior supports by creep deformation must be provided for. This was accomplished by suitable reinforcing details in one of the two girders.

(7) Fatigue Tests

Although the strength of the composite structure was thoroughly studied for static loads earlier in this investigation, it was considered desirable also to study repeated load effects. As seen in Fig. 11, the half-scale specimens used simulated that part of a girder taken from the design study bridge which extends 20 ft. either side from the center support. The only variable involved in these fatigue tests was the maximum value of the pulsating load. The specimens were supported at the diaphragm and were loaded at both ends by pulsating rams. It was found that a composite connection, designed to have a static ultimate flexural strength 2.5 times the service load moment, can sustain safely over

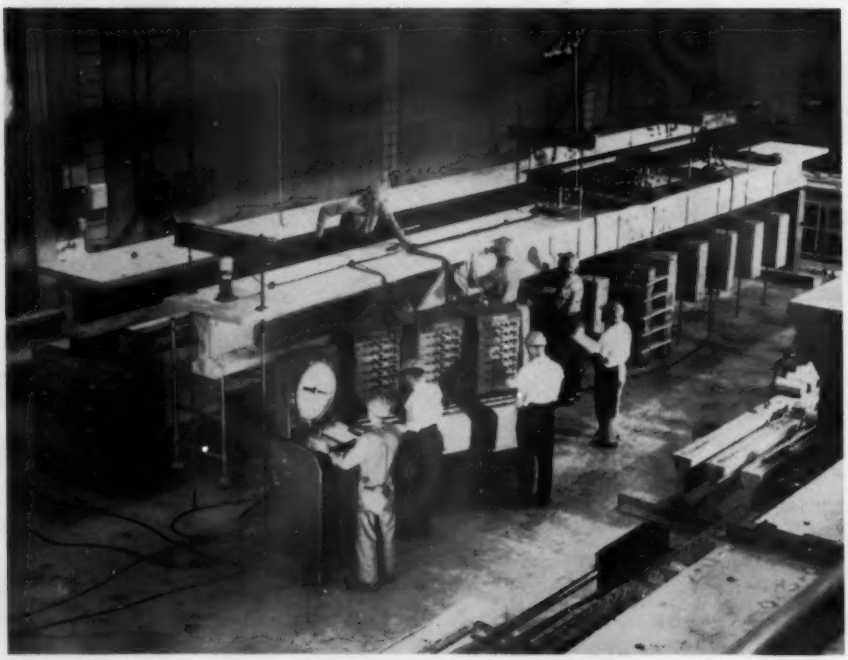


Fig. 8—Diagonal tension test of composite girder.

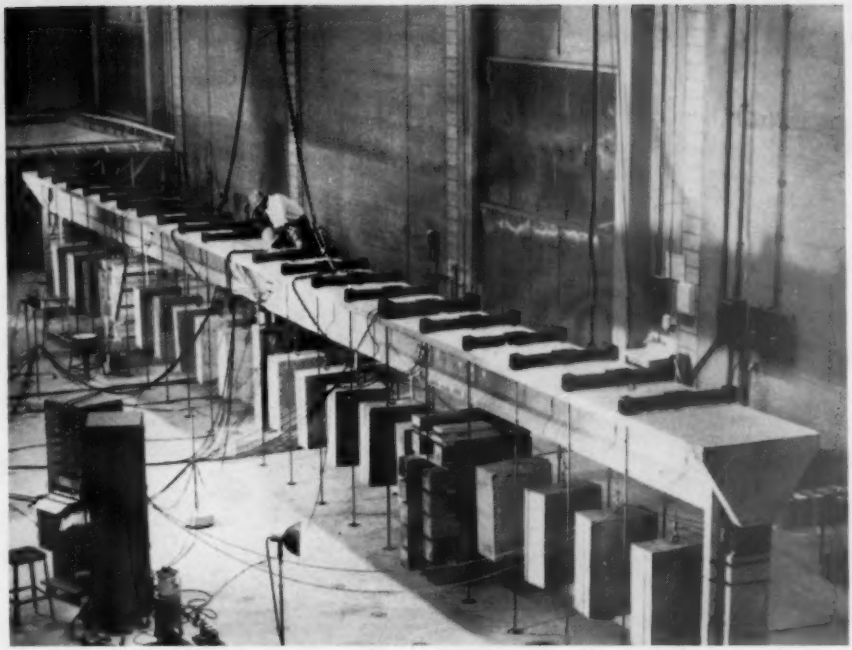


Fig. 9—Flexural test of continuous composite girder.

10 million repetitions of load to 1.5 times the service load.

(8) Reverse Bending

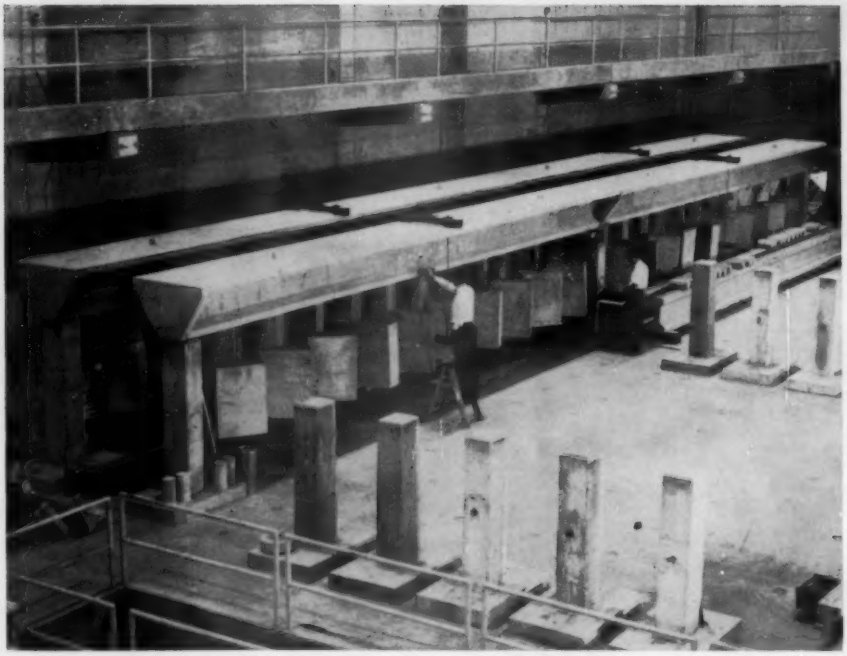
In continuous bridges with more than two spans, small positive moments may develop at interior girder support sections under some conditions of loading. Static and dynamic reverse bending tests were therefore carried out to develop suitable reinforcing details resisting such moments.

(9) Bridge Test

The final project of this investigation was testing of a complete half-scale, two-span continuous bridge. Two types of tests took place. Tests were first made at service load level to study the continuity behavior of the composite structure, and also to study the transverse distribution of concentrated load effects. Such a service load test is seen in progress in Fig. 12. Secondly, overload tests were carried out, culminating in a test to destruction under an arrangement of four concentrated loads, simulating the wheel loads of the extraordinary military loading, considered in design of bridges for the inter-state highway system. In all of these tests, extensive measurements were made of reactions, deflections and strains to permit comparisons between the observed behavior of the bridge and various theories.

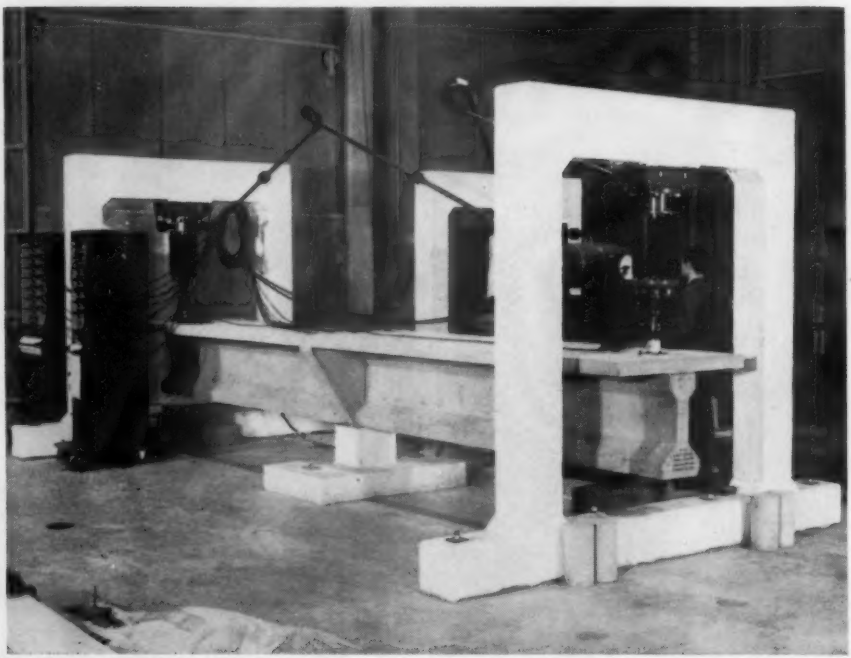
Publication of Results

The methods and findings of this investigation of composite bridge construction are being published in a series of Portland Cement Association Development Department Bulletins. The first two parts of the series are in press[12,13], and other parts will follow during 1960.



ABOVE—Fig. 10—Continuous composite girders under sustained load.

BELOW—Fig. 11—Fatigue tests of continuity connection.



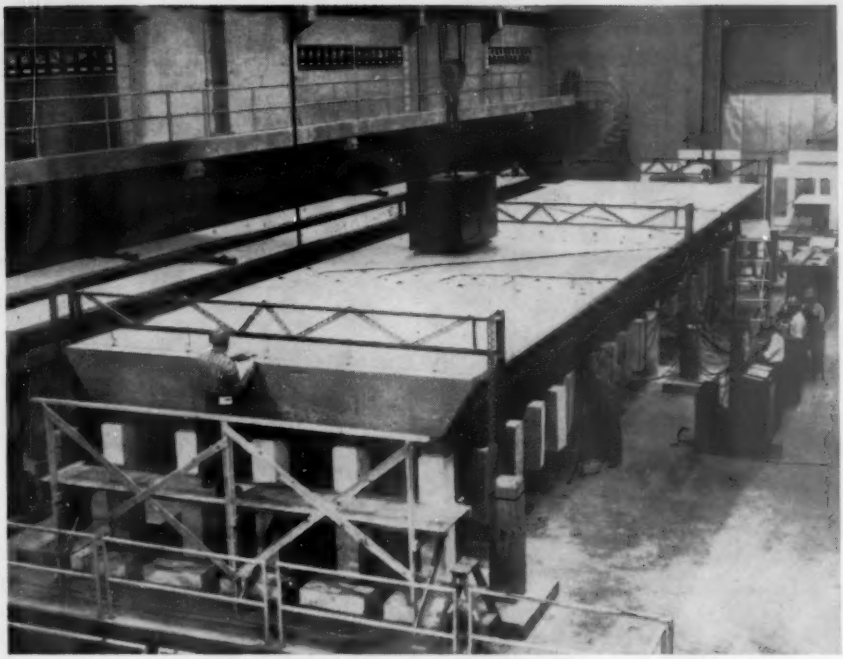


Fig. 12—Service load test of continuous composite bridge.

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CONCLUDING REMARKS

An extensive investigation carried out in the new Structural Laboratory of the Portland Cement Association concerns composite construction in which combined usage is made of precast-prestressed bridge girders and a conventionally reinforced situ-cast deck slab. It is expected that the results of this experimental and analytical study will contribute importantly to the science and art of prestressed bridge design.

Aside from their application to the specific bridge problems involved, the results of this investigation also contribute toward improvements in design and construction of composite building structures. To develop such application fully, a new investigation of similar scope is being planned for execution in 1960. Emphasis will be placed on composite buildings consisting of precast columns, girders and double-tee slabs, all connected together into an integral continuous structure by a situ-cast floor topping containing deformed bar reinforcement.

Bundled Strand | Its Effect on the Performance of Prestressed Beams

by: **Ross H. Bryan***
Consulting Engineer

This paper describes the results obtained from testing 24 beams with bundled and spaced strands. The prestressing force was maintained constant and "d" was varied in 4 x 11½ in. specimens. Four test series were made, having four values of "d", with three spaced strand specimens and three bundled strand specimens made for each series. The resulting influence on ultimate strength, cracking load, crack pattern, and deflection is tabulated and discussed. The test and computed ultimate strength loads of the beams are compared, using several current methods of computing ultimate strength.

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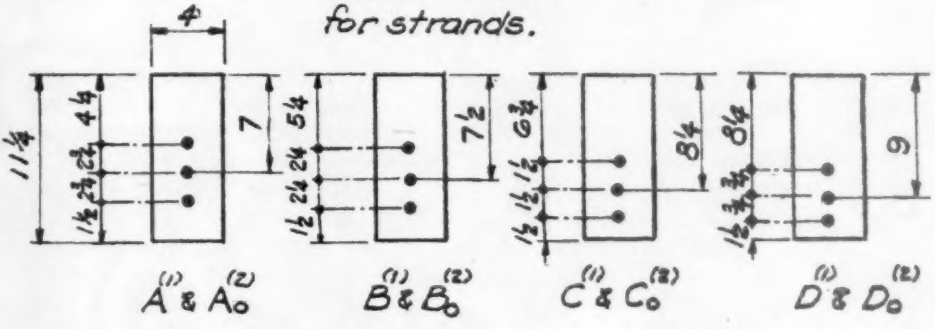
Description Of Test Beams

The beams were made in the fabricating plant of the Nashville Breeko Block Co. by members of the author's staff. All specimens were 4 in. wide by 11¼ in. deep. Three 5/16 in. diameter strands, bundled or spaced, were used in all specimens. See Fig. 1. The strands were tensioned by means of a center hole jack, jacking against the ends of 12 in. channel side forms. Tension in the strands was measured both by gauge and by elongation. In order to allow for the slip in the anchorages, the strands were over-tensioned slightly, to obtain the finally desired stress. A split washer was placed under the head of the anchorages to take up

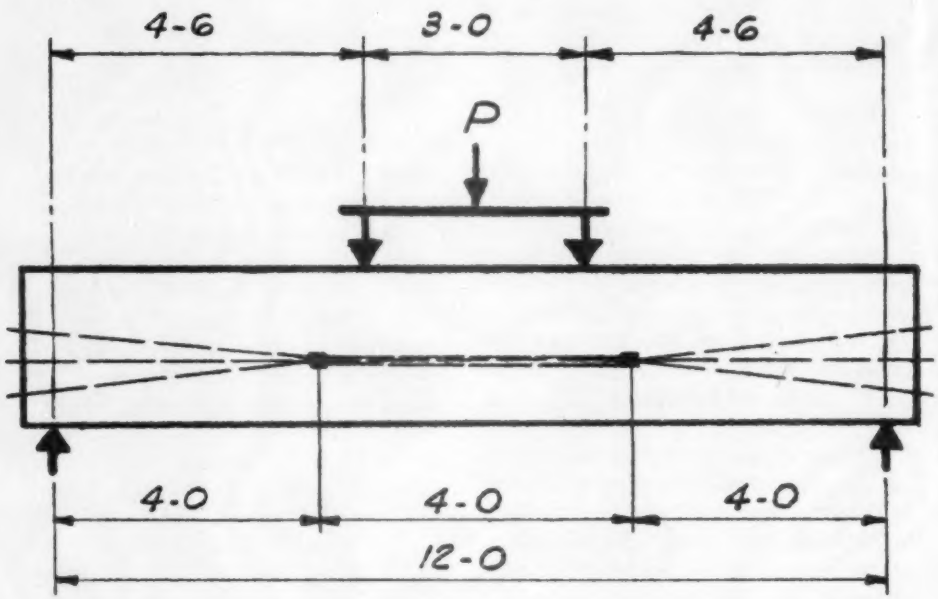
FIGURE No. 1 - Details of Test Beams and end anchorage dimensions for strands.



(1) Strands straight

(2) Strands bundled, see detail FIG. 2. All strands are 5/16" diameter

FIGURE No. 2 Showing testing setup and details of bundled strands.



the major portion of travel by the jaws.

Bundling of the strands was accomplished by running the strands through a pipe sleeve, which held them in contact with each other at the desired points. Fig. 2 shows the detail used in the fabrication of all bundled strand specimens. The strands were bundled through the middle third portion of the beam and spread at the exterior third of the beam, to the same anchorage point used in fabricating the companion spaced strand specimen, which had an identical c.g. of strand as the bundled strand specimen.

Fabrication of the beams extended over a period of four months, since the work had to be fitted into current work loads in the office, and was sometimes interrupted by inclement weather. The concrete used in the test beams was the concrete being poured by the plant in its daily production. The beams were left in the steel channel form for periods of from one to four days, depending upon the prevailing temperature and availability of man power. Immediately after the beam was made, it was covered with a curing paper and kept protected from the sun and rain. Heat was not applied as a curing accelerator.

As the beams were completed, they were stockpiled in a storage yard. Their age at the time of testing varied from one to five months. Testing of the beams was carried out at the laboratory of the Tennessee Agriculture and Industrial State College. The beams were tested on a 12 ft. span with two point loading,

as shown in Fig. 2. Load was applied 6 in. inside of the point of bundling, in order to confine cracking to the mid-third of the beam. Test results show that the principal tensile cracks were formed in this area. The crack pattern was marked on each beam, and a photograph of each test specimen was made after testing.

Table 1 shows a tabulation of the test specimens. The details of strand spacing and location are shown in Figs. 1 and 2. For each beam series, the end anchorage location and spacing of strands was identical, for the spaced strand specimen and the bundled strand specimen; the only difference being the bundled strand in the mid-third of the beam.

Tabulation Of Test Results

Table 2 shows a tabulation of the test results and includes the average compressive strength of each beam series at the time of testing, the ultimate load required for failure, and the estimated cracking load arrived at by studying the deflection curves and observing and listening to the beam. It will be noted that the concrete strengths varied from a minimum of 6200 to a maximum of 7470 lb. This is about the usual variation to be expected in a prestressed concrete fabricating plant, where high strength concretes are required in short curing periods. In evaluating cracking and ultimate loads, this variation in concrete strengths was considered, although in the case of ultimate load, it had little effect.

The recorded ultimate loads obtained in the test are compared to computed ultimate loads in Table 3. Assumptions used in computing the ultimate loads are shown at the bottom of the table. In an attempt to make an allowance for the variation in concrete strength, adjustment of test data is made in Table 4, for cracking and ultimate loads. It was felt that the cracking strength of a beam would be approximately proportional to the ultimate strength of the concrete, other things being equal. At least this was the thinking

TABLE NO. 1—TABULATION OF TEST BEAMS

TYPE	NO. BEAMS	SIZE b x t	STRAND	Depth "d" to c.g. of Strand	Strand Arrangement
A	3			7.00	
B	3	All Beams 4 x 11.25	3-5/16 All Beams	7.50	All Spaced
C	3			8.25	
D	3			9.00	
Ao	3			7.00	
Bo	3	All Beams 4 x 11.25	3-5/16 All Beams	7.50	All Bundled
Co	3			8.25	
Do	3			9.00	

TABLE NO. 2—TABULATION OF TEST RESULTS

TYPE	CRACKING LOAD Pc	Av. Pc	ULTIMATE LOAD Pu	Av. Pu	AVERAGE f'c
	6,000		10,660		
A	5,500	5,960	10,200	10,233	6,700
	6,380		9,840		
	6,400		11,200		
B	5,810	5,970	9,880	10,427	6,312
	5,700		10,200		
	6,200		12,280		
C	7,020	6,657	12,400	12,443	6,930
	6,750		12,650		
	7,250		13,500		
D	8,150	7,517	13,980	13,566	7,445
	7,150		13,220		
	5,840		9,980		
Ao	4,800	5,197	9,100	9,460	7,470
	4,950		9,300		
	5,500		10,760		
Bo	6,160	5,903	11,500	11,120	7,183
	6,050		11,100		
	6,000		11,800		
Co	5,970	5,990	12,000	11,753	6,200
	6,000		11,460		
	8,280		13,300		
Do	8,450	8,010	13,700	13,333	6,658
	7,300		13,000		

in determining the coefficient used to modify the cracking loads. In the case of cracking loads, the adjustment coefficient is obtained by dividing the average crushing strength of the concrete in the bundled strand series by the average strength of the concrete in the spaced strand series. This gives a modified coefficient to the bundled strand series, and a coefficient of one for the spaced strand series. In other words, the concrete is adjusted to the strength of the spaced strand series.

In computing the coefficient C_2, which modifies the ultimate strength of the beams, the computed value of J_d was used. Again, the adjustment was made to the bundled strand series, using the spaced strand series as a base, with a coefficient of one. The modification of the ultimate loads originated in the computed value of K_d based upon the crushing strength of the concrete. The corrected values of K_d were used in computing the final value of J_d, and the ratio of the J_d's thus obtained were used in computing the coefficient C_2.

Discussion Of Test Results

The values of $C_1 P_e$ are plotted in Fig. 3, to show the relation between

the cracking strength of beams of equal depth with spaced and bundled strand. A study of the curves shows that in the smaller depths, the spaced strand had a higher cracking value, and in the upper ranges of depth the grouped or bundled strand had a higher cracking value. It is possible that the large area of concrete below the strands in the lower depth ranges affected the cracking load. This must surely be the case in the spaced strand curve, where the cracking values for the 7 in. and 7½ in. depths are identical. In the upper ranges of depth, the bundled strand appears to give better results, as far as cracking is concerned, but a study of the photographs will show that the cracking pattern of spaced strand is more desirable.

It was observed during the testing of the beams that the spaced strand specimen appeared to have smaller, more closely spaced cracks, and tended to perform more uniformly. Figs. 9 through 12 show the photographs of two test series for both spaced and bundled strands. A study of these photographs will show that the spaced strands specimens have more regular cracking patterns and, what is probably more important, less tendency to slope off in the outer areas and become shear cracks. The cracks just outside of the load points in both of the bundled strand specimens show marked degrees of slope and an indication that a shear compression failure was imminent. It should be stated, however, that a shear compression failure did not develop in any of the test specimens. Compression failure always occurred at the center of the beam.

The adjusted ultimate load values are plotted in Fig. 4. It is rather

TABLE NO. 3—COMPARISON OF CALCULATED P ult. TO TEST P ult.

	TYPE	d	f'c	Calc. P_u	Test P_u	Test P_u/Calc. P
Spaced Strand	A	7.00	6,700	9,440	10,233	1.085
	B	7.50	6,312	10,100	10,427	1.033
	C	8.25	6,930	11,500	12,443	1.085
	D	9.00	7,445	12,750	13,566	1.062
Bundled Strand	Ao	7.00	7,470	9,559	9,460	0.992
	Bo	7.50	7,183	10,300	11,120	1.081
	Co	8.25	6,200	11,300	11,753	1.040
	Do	9.00	6,658	12,600	13,333	1.059

$$Mu = Tu \times Jd$$

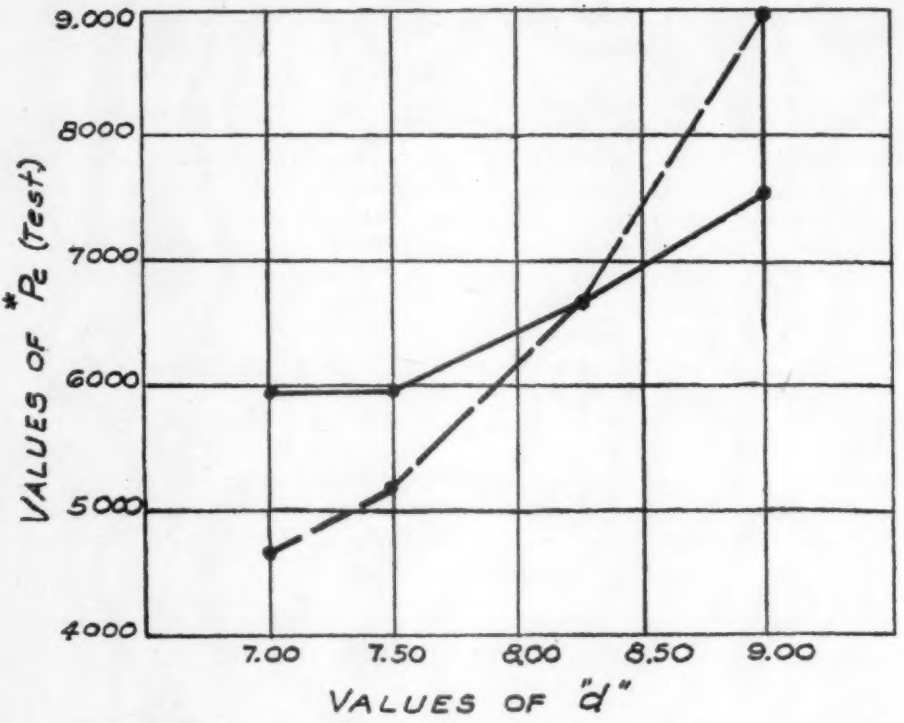
$$Tu = 3 \times 14500 = 43{,}500\#$$

$$Jd = d - \frac{Kd}{2}$$

$$Kd = \frac{43{,}500}{4 \times .85 \times f'c}$$

$$P_{calc.} = \frac{Mu\ calc. - DLM\ Beam}{2.25}$$

FIGURE 3-Cracking loads for spaced and bundled strand with Tu constant and "d" variable.



*Corrected for concrete strength, see Table No.4

—— Spaced strand

--- Bundled strand

apparent that these values show the typical spread of rather uniform test data. A line drawn through the middle of the points on the curve would lie almost equidistant from all points. It is therefore concluded that the ultimate strength of the beams was not affected by handling of the strands. It should be noted here that the test data covers a very wide range of ratios between the total strand spread and the total effective depth of the beam to the c.g. of the strand. In the case of beams with a 7 in. "d", this ratio is 0.79 and in the case of beams with a 9 in. "d", the ratio is only 0.167.

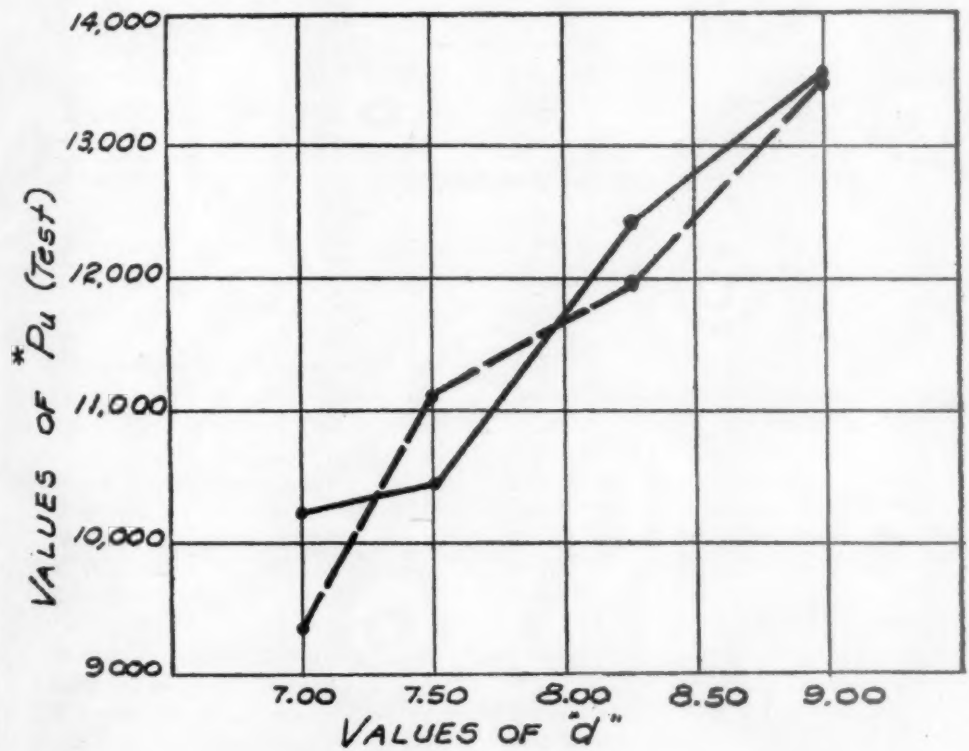
In designing the test series, it was felt that the wide spacing of strand would effect the ultimate strength unfavorably, because of the greater elongation required of the bottom strands, in order to build up to the ultimate capacity of the strand. It was felt that in a relatively short, deep beam that this would be espe-

cially apparent.

Figs. 5 through 8 show the typical load deflection curves of Type B and Type D test series. A study of these curves will show that the spaced strands gave a smaller deflection at all stages of loading. This is also contrary to what was expected. It would appear that the spaced strands, by holding the concrete deformations to a minimum, materially increased the effective moment of inertia of the beam and thus reduced the deflection and increased the cracking load.

The deflection curves show that both the B and D series were loaded and unloaded five times. The fifth and final loading being carried to failure in each case. The first loading in each case being carried to approximate cracking load, but it

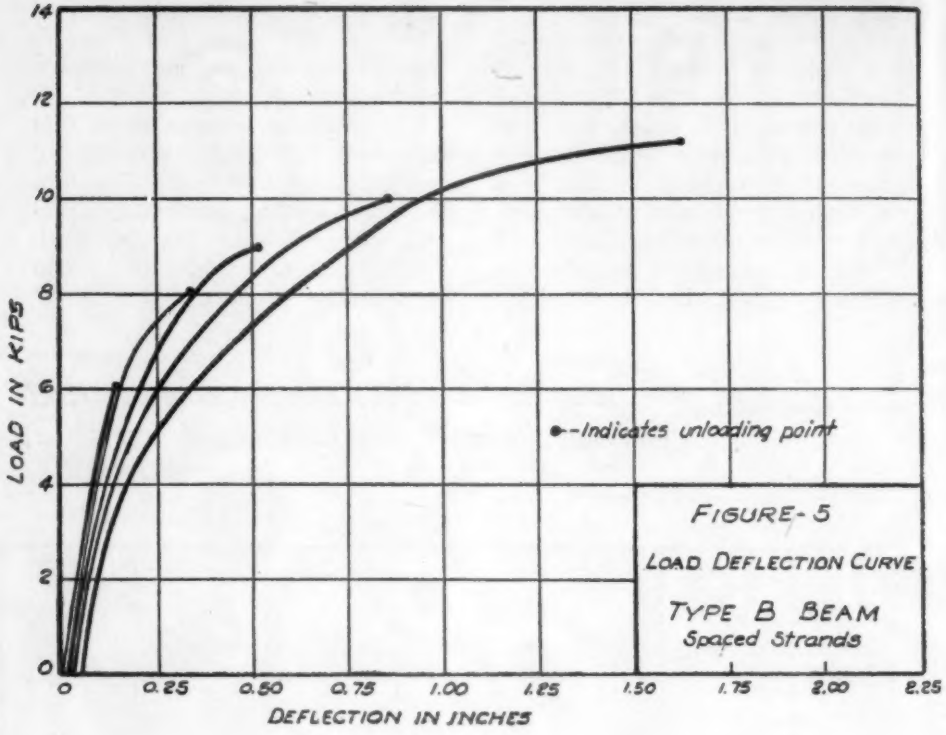
FIGURE 4 -Ultimate loads for spaced and bundled strand with Tu constant and "d" variable.



*Corrected for concrete strength, see Table No. 4

—— Spaced strand

--- Bundled strand



was intended that the load should be removed just before cracking. This was the case in most of the beams.

The curves show that on the fourth loading, when the beams were loaded to approximately 90 per cent of their capacity, recovery was also at least 90 per cent and in some cases, even greater. There appears to be no measurable difference between the recovery of a spaced strand specimen and a bundled strand specimen.

Stress In Strand At Ultimate Load

In computing the ultimate loads listed in Table 3, the ultimate stress in the strand was assumed to be the minimum published ultimate strength, or 14,500 lb. for each strand. The ratio of test to calculated loads indicates that this stress was exceeded in every case but one. The variation ranges as high as 8-1/2 per cent, with an average of about 5-1/2 per cent. This would indicate that the value for the ultimate stress in the strand at the time of failure is approaching the actual value of the ultimate strength of the strand. It is recognized that the published minimum strengths are approximately 10 per cent under the actual breaking strength of the strands of most manufacturers.

It has been proposed by some investigators that the stress in the steel used in computing ultimate strength of a prestressed beam be based upon the following formula, which includes the percentage of steel as a variable. $fsu = f'c\ (1\text{-}0.5\frac{pf's}{f'c})$. This

test series had a variation in percentage of steel from 0.482 to 0.620, or a variation of approximately 30 per cent, and it is significant that no influence on the ultimate steel stress due to this variation in steel percentage could be detected in the test results. For 6500 lb. concrete and a percentage of 0.62, the calculated steel stress at ultimate load by the above formula gives a total load of 38,200 lb. for 3-5/16 in. strands. For a steel percentage of 0.482 and a concrete stress of 6500 lb., the computed failing load for 3-5/16 in. strands is 39,300 lb. Since the test results indicate that the beams failed at an average steel stress approximately 5 per cent greater than the stress giving 14,500 lb. per strand assumed in the computations, we have a failing load in the 5/16 in. strands in the average test specimen of approximately 15,300 lb. per strand. The 15,300 lb. stress per strand gives a total force of 45,900 against the total force computed by the above formula of 38,200 for a percentage of 0.62; and 39,300 for a percentage of 0.48. This is a variation of 20 per cent and 17 per cent respectively, which is a considerable variation and one which may be overly conservative.

It is recognized that the percentages of steel used in the tests were in the low range, but the percentage of the Type A beams was approaching, at a value of 0.62 per cent, the amount of steel which should be used in a design in which sudden failure is to be avoided. The test series was designed to get a maximum load from the steel and a delayed concrete failure.

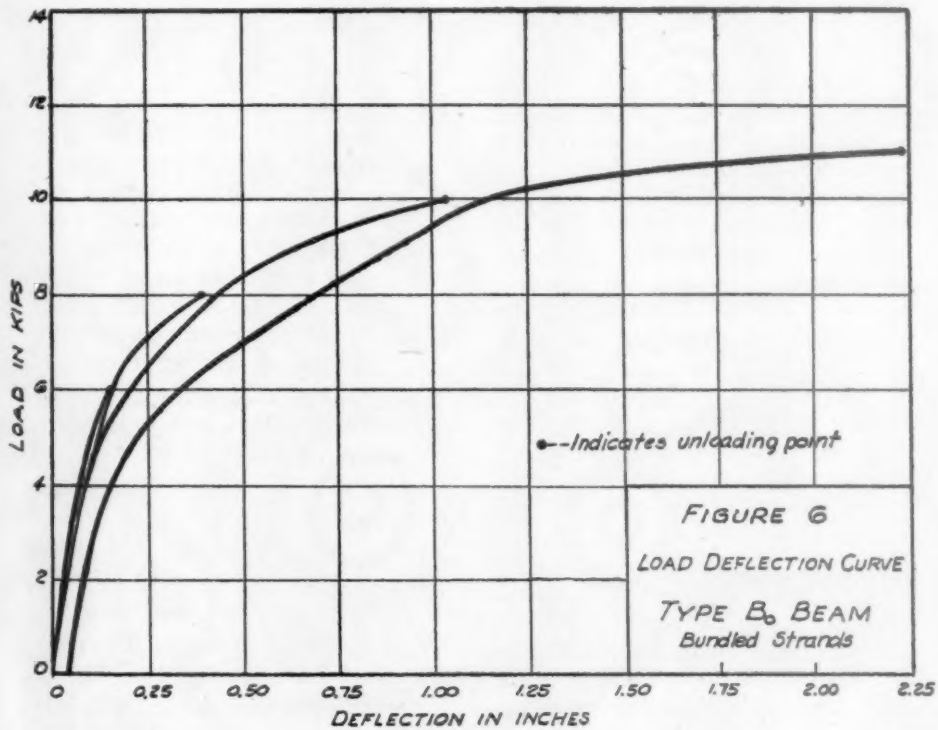
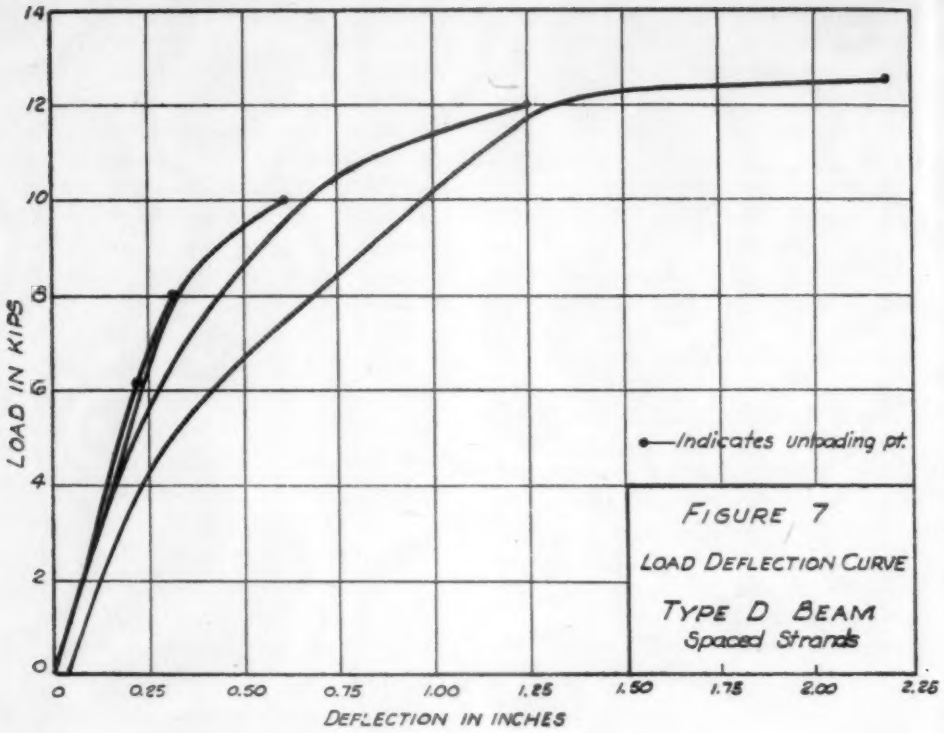


FIGURE 6
LOAD DEFLECTION CURVE
TYPE B_0 BEAM
Bundled Strands



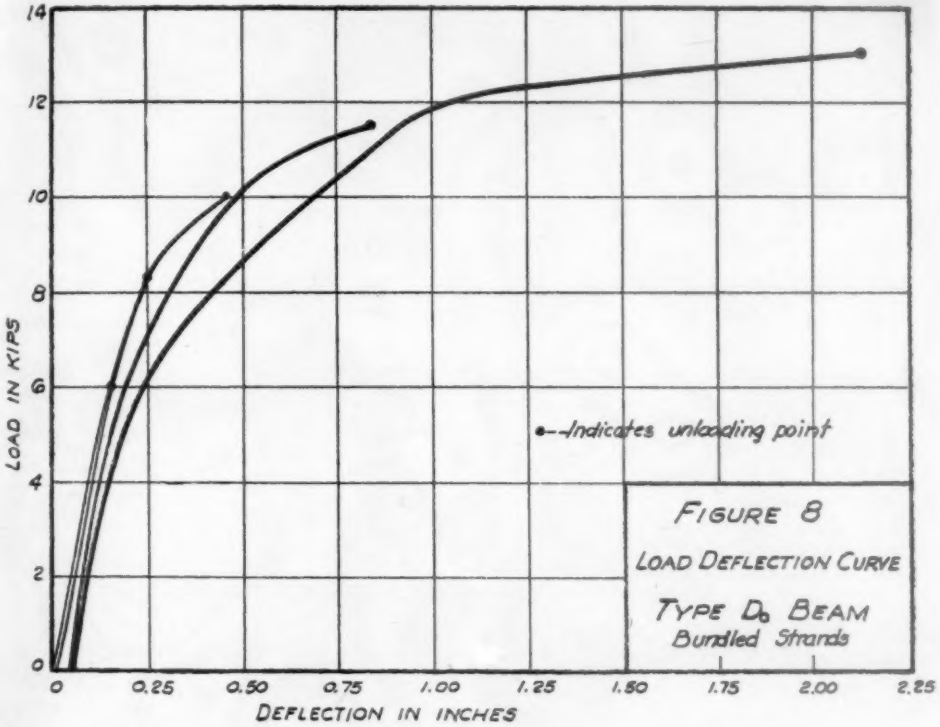
Shear

A study of the ultimate failing loads for all beams, and through the ranges of all depths, shows that the average shearing stress at ultimate load was approximately 215 psi when computed with the formula $v = \frac{V}{bjd}$. There was no shear reinforcement in the beams. The crack pattern indicates that a shear compression failure was probably approaching at the load points of the beam, but as previously stated, it did not occur there.

CONCLUSIONS

It is recognized that additional test data should be obtained for higher steel percentages, but it appears that some conclusions can be drawn from the test data presented. There appears to be no measurable difference between the ultimate strength of a beam with bundled strand and a beam with spaced strand, even when the spacing of the strand extends over a distance equal to 0.90 of the effective "d". While the performance of the spaced strand under cracking and deflection, especially in the higher load ranges, appears to be slightly better than the performance of beams with bundled strand, the performance of both types at working loads is almost identical and the performance above working loads does not show sufficient variation to warrant the selection of one over the other. The most important factor brought out by the tests is the apparent ultimate

strength obtained in the strand at failure. For the average design, it appears that the published ultimate strength of the strand can be used with safety when the beams lie within the under reinforced range.



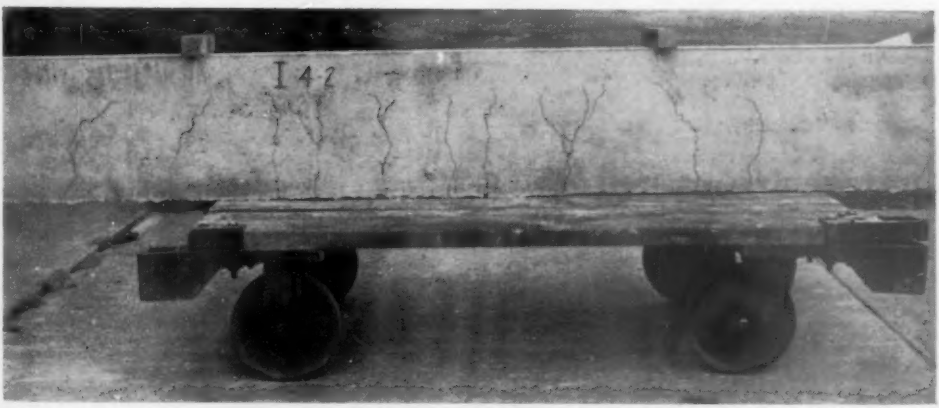


Fig. 9—Series B. d = 7.50" Strand Bundled.

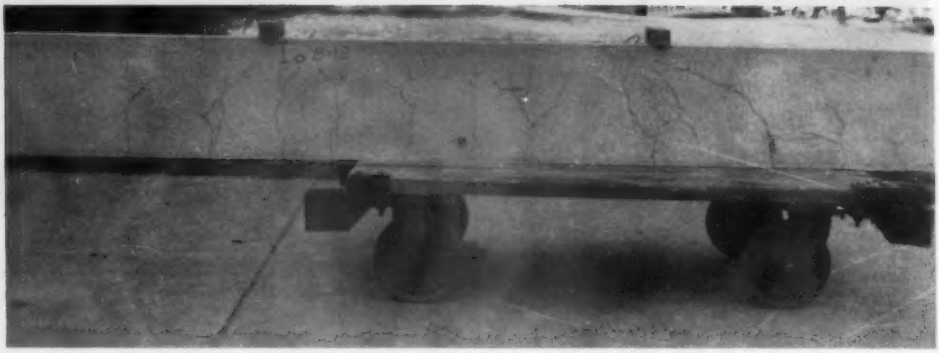


Fig. 10—Series Bo. d = 7.50" Strand Bundled.

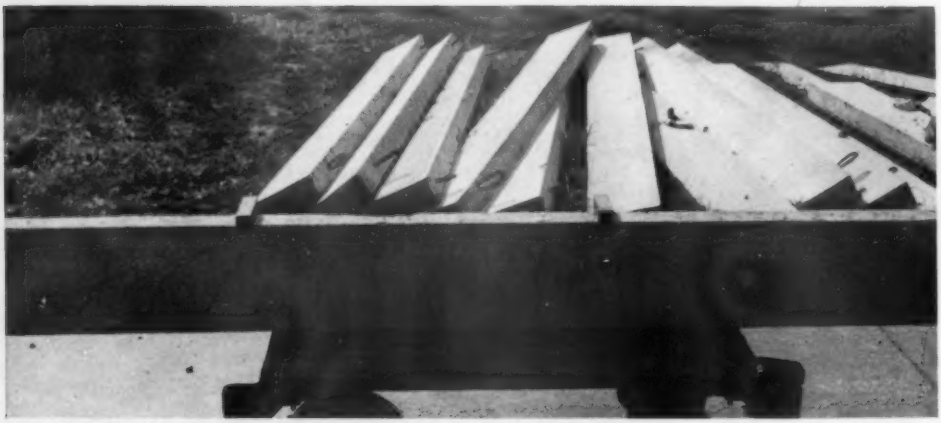


Fig. 11—Series D. D = 8.25" Strand Bundled.

Fig. 12—Series Do. d = 8.25" Strand Bundled.

TABLE NO. 4—ADJUSTMENT OF TEST DATA FOR VARIATION IN CONCRETE STRENGTH

TYPE	f'c[1]	C_1	Pc[1]	C_1 Pc	C_2	P. ult.[1]	C_2 P ult.
A	6,700	1.00	5,960	5,960	1.00	10,233	10,233
Ao	7,470	0.90	5,197	4,680	0.988	9,460	9,350
B	6,312	1.00	5,970	5,970	1.00	10,427	10,427
Bo	7,183	0.88	5,903	5,200	1.00	11,120	11,120
C	6,930	1.00	6,657	6,657	1.00	12,443	12,443
Co	6,200	1.12	5,990	6,700	1.015	11,753	11,950
D	7,445	1.00	7,517	7,517	1.00	13,566	13,566
Do	6,658	1.12	8,010	8,980	1.012	13,333	13,500

[1]All values average of three test beams.

$$C_1 = \frac{\text{f'c of spaced strand}}{\text{f'c}}$$

$$C_2 = \frac{\text{Jd of spaced strand}}{\text{Jd}}$$

Readers' Comment

Karl H. Middendorf, Chairman of the Board, of the Prescon Corporation, was a pioneer of prestressing and a key figure in its development. If you don't know him you have missed meeting a philosopher, an engineer of high intellect. Naturally any paper submitted to this editor by Karl Middendorf would always receive prime consideration. Before me is one which he believes would be of interest to the readers of the JOURNAL. It covers a subject which I have found to be a theme with strong controversial differences of opinion. I have therefore opened up the "Reader's Comment" section to this distinguished pioneer with his personal permission to invite criticism. He very graciously says—"Sure. Perfection comes from discussion."

First, let me throw in my two bits worth of criticism. Stress-relieved wire is many times more resistant to stress-corrosion than prestretched wire. The modulus of elasticity of prestretched wire varies considerably from one wire to the next making it almost impossible to tension a group of wires simultaneously with any degree of accuracy. At working loads the relaxation of stress-relieved wire is less. Wire with an appreciable curvature cannot be used for making cables but mechanical straightening destroys the desired elastic properties. Stress-relieving by heat restores these properties but prestretching does not. Prestretching does little to improve the proportional limit.

From here on out I invite others who differ from Karl Middendorf to carry the ball because the subject is important to the industry.

M.P.K.

Comparison of Cold-Drawn Stress-Relieved Wire

By Karl H. Middendorf
Chairman of the Board
The Prescon Corporation

Ever since high strength cold-drawn wires have been used as components of prestressed tendons, there have been advocates of stress-relieving through heat treatment and stress-relieving through over-stressing held for a limited period. Generally it has been stated that over-stressing for a period of two minutes would modify the stress-strain relationship so that the cold-drawn wire would approximate the proportional limit, 0.1 per cent offset and 0.2 per cent offset of stress-relieved wire.

Our company in the manufacture of its custom prefabricated tendons, could with minor modification of its equipment, produce not only one elongation of two minutes but any reasonable number of elongations for periods up to three or four minutes. To determine if comparable results could be obtained with a wire costing less due to elimination of the heat treating process, we ran the comparative tests tabulated.

To establish reasonable comparisons, wire was obtained from the mill from the same heat and from the same roll as it came from the last draw. Part of this wire was heat-treated through immersion in a lead bath. The stress-relieved wire and the cold-drawn wire were tested as received. In addition samples of the cold-drawn wire were stressed through a 3-in. pipe. The wires were cut to an exact equal length and headed to assure positive uniform length. Elongation required to produce a stress of 180,000 psi was calculated based on a modulus of elasticity of 24 (10^6). In stressing, the equipment was set up so that a fixed steel shim was inserted to maintain the elongation for the required period. The period of rest between each elongation was the same as the hold period. After the wires were stressed as indicated in the tabulation, they were cut into three equal pieces and headed at each end with a 3/4-in. diameter washer under each head and marked with identifying numbers. These positive anchored samples were used by the laboratory to furnish stress-strain curves and other information used in developing the data listed in the table below. Since the various samples came from approximately 300 ft. of the same draw, it was assumed that the cross-sectional area would be uniform throughout. (See accompanying tabulation.)

The difficulty of establishing the point where the stress-strain curve of high strength wire departs from a reasonably straight line is well known. Care was taken to endeavor to locate this point as it is felt it is quite important in the long life of stressed wire in concrete. It is recognized, however, that these read-

ings could vary 500 psi plus or minus.

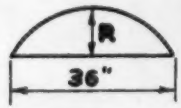
The data indicate that each successive elongation raised the elastic properties of the wire. Also the properties seemed to be enhanced by the length of time the elongation was held. Projected, it would appear that four elongations for a period of three or four minutes each, might closely approximate the properties of stress-relieved wire at 0.2 per cent offset and reasonably at 0.1 per cent offset.

However, in the area of the proportional limit it does not appear that cold-drawn wire will come within 20,000 psi of the stress-relieved wire. Regardless of the fact that wire specifications rarely point to this limit as a guide, preferring the offset rules for accuracy, tests indicate that fixed loss due to creep, shrinkage, etc., do not fully occur within periods of less than two years. With a design stress of 144,000 psi based on 0.6 of a guaranteed ultimate of 240,000 psi it is possible that 10,000 to 15,000 psi of working stress may be lost.

It would appear that if cold-drawn wire is to be used the working stress should be held to a maximum of 0.50 to 0.55 of the ultimate instead of 0.6 used for stress-relieved wire depending on the number and length of elongations. In addition, it would appear that the single elongation for two minutes is insufficient and should be increased not only as to time but as to the number of repetitions. It is expected that the percentage of elongation in 10-in. would be considerably below that of the stress-relieved wire. In this respect the cold-drawn wire proved ample in all cases to meet the most rigorous specifications.

Although the cold-drawn wire was received in approximately 36-in. coils and thus had a distinct curvature, it was hoped that the prestraining would tend to straighten it. The tabulation shows that this was a false hope. Apparently there is no way to have a straight wire except to have it straight and cooled before coiling in 6-ft. rolls at the mill. Straighteners can be used on cold-drawn wire prior to elongation. Straightening of any wire, however, after the proportional limit has been increased either by heat-treating or elongation will drop the proportional limit almost to the original value of the cold-drawn wire. This has been demonstrated by redrawing wire after stress-relieving. Where straightening, acute bending, or drawing are required as a part of the stressing operation, there is no virtue in using stress-relieved wire. The logical procedure is to reduce the working stress as suggested.

Considering the reduced stresses which should be used with cold-drawn wire in relation to the cost difference between cold-drawn and stress-relieved, there does not appear to be an economic advantage in its use, except where stressing procedures would invalidate the values gained by stress relieving through heat treatment.

Type of 0.25" Dia. wire used in the test	Type of pre-strain by fixed elongation to approximately 180,000 PSI		Proportional limit *	Stresses in wires under test as developed from the stress-strain curves			Percentage elongation in 10" gauge length at rupture	Modulus of Elasticity X 10^6	Final Curvature of wire
	No. of Times Elongated	Minutes held under each elong.		at 0.1% offset	at 0.2% offset	at ultimate stress			 R in inches
cold-drawn stress-relieved	0	0	177,600	199,400	206,800	245,700	6.5	28.94	.01
cold-drawn untreated received in approx. 36" coils	0	0	146,400	172,600	188,500	247,400	5.6	22.13	.31
	1	1	147,200	180,500	193,500	246,400	5.3	24.84	.30
	1	2	147,600	181,100	194,500	247,500	5.0	24.37	.30
	1	3	147,900	181,600	194,800	247,500	5.2	23.69	.30
	1	4	148,300	181,900	195,500	247,400	5.6	24.45	.29
	2	1	146,500	181,500	197,400	248,600	5.5	24.74	.30
	3	1	148,400	184,900	200,300	249,400	5.4	23.69	.30
	4	1	150,300	186,500	202,600	248,600	5.2	23.89	.29
	5	1	151,500	187,200	203,800	247,900	5.3	24.31	.28
	6	1	152,500	187,400	204,100	248,400	5.6	25.80	.28
	2	2	148,500	183,200	198,500	247,900	5.7	26.50	.29
	3	2	149,100	185,400	201,300	248,500	5.6	24.66	.29
	4	2	149,300	188,500	203,800	249,300	5.2	23.04	.28
	2	3	154,300	184,700	198,700	247,400	5.1	24.37	.28
	3	3	154,400	186,200	201,800	247,500	5.0	26.29	.27
	2	4	154,500	185,500	208,200	248,500	5.3	24.02	.27

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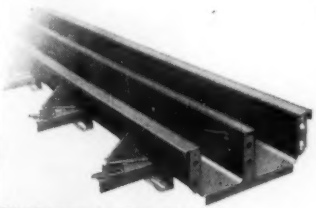

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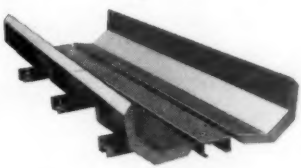

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